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ORDINARY MEETING.

2 November, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., the retiring President, in the Chair.

Sir ALEXANDER GIBB said that it was, he knew, with great regret that the members of The Institution had heard of the deaths, during the recess, of the Marchese Marconi, Honorary Member, and of Dr. H. H. Jeffcott, Secretary of The Institution. Resolutions of condolence had been passed by the Council and had been sent to their respective families, and he asked those present to stand for a short period as a mark of respect.

It was now his great pleasure and honour to introduce—although no introduction was necessary—the President for the coming year, Mr. Sydney Bryan Donkin, and to congratulate him and invite him to take the Chair.

Mr. SYDNEY BRYAN DONKIN, President, having taken the Chair,

Sir CHARLES MORGAN, Past-President, moved the following resolution :—

> "That the members present at this meeting desire, on behalf of themselves and others, to record their high appreciation of the services rendered to The Institution by Sir Alexander Gibb during his term of office as President."

The resolution was one, he said, which was usually proposed by his old friend Sir Robert Elliott-Cooper. Sir Robert not being present that evening, the duty had devolved upon himself. He could not

add anything to the resolution, which expressed very well the feelings of the members of The Institution. On behalf of all those present, however, he would like to say that Sir Alexander Gibb had performed the duties of President in such a manner as to deserve the highest praise which they could possibily give him.

Mr. JOHN D. WATSON, Past-President, in seconding the resolution, said that all the members highly valued the work that Sir Alexander had done, and personally he appreciated it very highly, because he knew what a very arduous task it was that Sir Alexander had performed so admirably. It was a great pleasure to recall the fact that it was more than sixty years since they had first met. They had been born within a few miles of each other, and felt as though they were "brither Scots."

The resolution was carried by acclamation.

Sir ALEXANDER GIBB, in acknowledging the vote of thanks, thanked Sir Charles Morgan and Mr. Watson very much for their kind words, which he was afraid were largely undeserved, and said he was very grateful to the members present for the way in which they had received the resolution. He had had a very happy year as President, and, though naturally he left the Chair with regret, there were compensations, as Mr. Donkin would find when his own turn came to relinquish the Presidency. He wished to thank the members generally for their forbearance, courtesy, and kindness to him while he occupied the Chair, and to wish Mr. Donkin a very successful and happy year of office. The Institution could not have a better President than Mr. Donkin.

The President then delivered the following Presidential Address:—

GENTLEMEN,

Election to the Presidency of this Institution is not only an honour to which I am personally deeply sensitive, but is also an honour to the branches of Engineering in which my activities have been centred. Siemens, Kennedy, Preece and Bramwell, who were also in whole or in part connected with mechanical and electrical engineering, and who preceded me in this Chair, have left a considerable gap for me to cover. I shall attempt, however, to carry on the story that Kennedy and Siemens told you some 30 years ago and to refer more specifically to the particular problems that have impressed

me most during the period of my Corporate Membership of The Institution.

The progress of the application of electricity for the use of man was slow up to the beginning of the last century. It is probably known to all of you how it is reputed to have started with Thales about 600 B.C., and how it had only been carried on very intermittently since then. One can recall the work of Gilbert in 1600, of Franklin about 1750, of Wimshurst and Volta about 1800, and then of Faraday, who in 1831 made the fundamental discovery of electro-magnetic induction which proved to be the foundation of the electric-power industry. In 1937, it is appropriate to refer to the centenary of the first use of the electric telegraph on English railways in 1837, due to Wheatstone and Cooke. Other names that stand out in my mind as famous in the progress of the science of electrical engineering include those of Humphry Davy, Gramme, Edison, Brush, Swan, Jablochkoff, Fleming, Ferranti, Hopkinson, Steinmetz, Parsons and Duddell.

Shelley wrote an essay on applied science about 1810, while still an undergraduate, and discussed the transmutation of elements and the uses of electricity. He said of electricity, "What a mighty instrument it would be in the hands of him who knew how to wield it, and in what manner to direct its omnipotent energies." He knew of Volta's invention of the galvanic battery and wondered what the effect would be if such a "new engine" were developed to a colossal magnitude. He referred with humour to the aerial mariner who swam in the air with bladders, and admitted that such ingenuity should not be condemned as it might lead to some future development then unconceived. "Why," he said, "have we not dispatched intrepid aeronauts to cross Africa in every direction to survey the whole peninsula in a few weeks. It is by these means," and he referred to all branches of applied science, "that we could advance civilization, emancipate every slave and improve generally the welfare of mankind."

Now most of the things Shelley thought of have been done and mankind has benefited. Man has become a more efficient unit; he performs more work in a given time; he lives longer; he has greater comforts; but, with all these advantages, it is a misfortune that the necessity still arises for some to spend their energies and the wealth of their country in fruitless endeavours to kill, or to be ready to kill, their enemy in less time and at less cost than is possible for their adversaries.

During the last 40 years we have seen the introduction of wireless, broadcasting and television, and we have learnt to appreciate their

effect on society. We have seen the development of the cinema, and of silent and of sound films, and their effect on all classes of listeners, especially on relatively uncivilized native peoples. We have lived to enjoy, or dislike, a great change in transport facilities, brought about by the automobile, and have realized the part that electricity has played in the progress of this branch of engineering. We have watched the introduction of new roads with their traffic-control systems, and have seen how this progress in automobile engineering has very largely altered our habits. Our ideas of warfare have been revolutionized by the great advance made in aeronautics, which has also made the world seem smaller in consequence of the great increase in the speed of travel. Wireless beams for directing aircraft to their ports and wireless communication have already helped to make for safety in flying. Even betting has been mechanized with the help of electricity, whilst the burglar finds it more difficult to carry out successfully his feats of daring. Photo-electric cells have not only made possible the detection of unwanted shadows, but have been responsible for the automatic illumination of lights at sea when daylight fails, and have made television and the sound film successful. The cathode-ray tube, too, has made television transmission possible with good definition, and has also been the means of analyzing physical phenomena depending on infinitesimal periods of time. Electricity has played a great part in the practice of welding which has so largely altered the methods of construction of plant and steelwork generally, while materially reducing its cost. The automatic piloting of ocean-going vessels and of aircraft has provided additional safety to these methods of transport.

The low-temperature carbonization of coal, the hydrogenation process for the production of light oils, and the means of producing other valuable by-products from such processes have very largely altered the chemical-engineering industry. The application of some of these systems to the generation of electricity has not developed as at one time was thought probable, but it is notable that in 1936 about one-tenth of all the fuel oil and motor spirit used was produced in Great Britain, and that the quantity is increasing.

Great progress has been made in the quality of materials for engineering work, and research at the National Physical Laboratory, organized by the Department of Scientific and Industrial Research, and in private laboratories, has guided engineers in the production and use of materials suitable for special conditions, notably at high temperatures, and has enabled greater progress to be made than otherwise would have been possible. The knowledge of the creep-factor and the limitation that it necessitates, together with the production

of alloys that have high limits of ultimate creep-stress, have enabled the efficiencies and the wearing properties of machines to be very greatly increased.

Perhaps, however, one of the most outstanding changes during the last 40 years has been the great increase in speed of all kinds with, at the same time, the means for controlling it.

Generation of Electricity.

During this same period the efficiency of the generation of electricity has been greatly increased. I find from the early records kept by Professor (later Sir Alexander) Kennedy of the steam generating stations of the Westminster Electric Supply Corporation, Ltd., that in 1891 the average consumption of coal was about 10 lb. per kilowatt-hour sent out, equivalent to a thermal efficiency of about 2·5 per cent., and that the works-cost of generating these units with good Welsh coal was about 5*d*. per unit.

The Electricity Commissioners' returns of fuel consumption for the year ending 31st December, 1936, indicate that the lowest coal consumption of any selected generating station in Great Britain is that obtained at the Battersea station of The London Power Company. This amounts to 0·97 lb. per unit sent out and is equivalent to a thermal efficiency of 27·63 per cent.

The approximate figure at the present time for the lowest cost of generating electricity in statutory selected generating stations, including capital charges, is of the order of 0·2*d*. per unit and in this connexion it is necessary to bear in mind that the generating stations with the highest efficiency are not necessarily the stations that are able to show the lowest cost per unit generated. Naturally the cost of coal, the capital charges, and the load-factor of operation, all affect the result considerably.

If all the electricity generated (by statutory authorities) in this country at the present time, now costing on the average 0·4*d*. per unit, were produced at this lowest cost of 0·2*d*., corrected for the average load-factor, the saving would be of the order of £9,000,000 per annum, which may be useful to consider as something to look forward to in the future.

The Weir Report of 1926 showed that the units sold by the Authorized Undertakers in Great Britain per head of population were then 110 per annum. It contained an estimate that this figure would increase to 500 units per head per annum in 1940. The actual figure for 1935–36 was 330, so that the estimated rate of increase has

been realized. The Report also contained an estimate that the total units sold per annum would rise to a figure of 21,000 million in 1940, and the average annual increase from 1926 to the present time shows that this estimate will undoubtedly be obtained. I think I am not unduly optimistic if I predict that by 1950 the total units sold in that year will be at least 30,000 million, whilst the units sold per head of population will be of the order of 600. Other conditions remaining reasonably constant, I think it possible to forecast that the average price of electricity will fall from 1·125*d.* per unit—the figure for 1935–36—to a little below 1*d.* per unit in 1950.

It is useful to trace the steps by which the present great improvement in the cost of steam-generation has taken place: firstly, by the mechanical handling of coal; secondly, by the more efficient combustion of coal; thirdly, by the utilization of all possible waste heat from such combustion; fourthly, by increasing the pressure and the temperature of steam to such limits as are economically sound and physically possible with materials available at the present time, to give the greatest possible heat-drop in the prime mover; fifthly, by the improvement in the efficiency of the prime mover together with the regenerative use of steam for feed-water heating; and lastly, by the improved efficiency of the condensing plant and of the electric generators themselves.

When referring to the improvement in the efficiency and to the lowering of the cost of the generation of electricity, it is essential to refer also to the advantage to be obtained in thermal efficiency by means of the internal-combustion engine. Even in 1892, the gas engine, when coupled to and driving a dynamo, was able to show an overall thermal efficiency of gas to electricity of about 14 per cent.; whilst the oil engine, at the same date and calculated in the same way, showed an efficiency of about 18 per cent. Diesel improved upon these results by means of his high-compression engine, and the overall efficiency of oil to electricity rose to a figure of about 25 per cent. Still, by the introduction of steam into the cylinder, produced by the extraction of heat from the exhaust, raised the efficiency of the diesel engine to the highest figure that, I believe, has yet been obtained, namely 39 per cent. This figure was, however, from oil to brake horse-power. The largest diesel engine and generator in an electricity power-station in 1936 was that erected a few years ago at Copenhagen, where the set has a normal output of 12,500 kilowatts, an overload output of 15,000 kilowatts, and the overall efficiency, on test at normal output, from oil to electricity is of the order of 36 per cent.

There has always been considerable controversy concerning the

advantages of the use of internal-combustion engines as opposed to steam-driven prime movers for the generation of electricity. There have been advocates for the use of the internal-combustion engine by the provision throughout the country of a large number of smaller power-stations. By this means it was proposed to reduce the cost of transmission, and to avoid the concentration of large blocks of plant in the more isolated steam stations. The solution of these problems is primarily one of cost, although it must also be one of political economy. A practical reason for using our own coal rather than imported oil is the consequent employment of more miners.

The Electricity Commissioners' latest returns show that the capital expenditures on generation per kilowatt of maximum demand for the smaller diesel-engine stations and for the larger steam stations are of the same order, but they also show that the working costs per unit sent out (or generated) are more than three times as great for small diesel stations as for the larger steam stations, in spite of the fact that the thermal efficiency of the diesel stations is about 50 per cent. higher. This indicates, amongst other things as far as public supply undertakings are concerned, that diesel generating stations for limited demands and for low load-factors, including peak-load stations, are fairly competitive with steam stations within the limit of fuel costs at the present time. The failure of the diesel stations to give low works-costs is, of course, due to the high cost of fuel oil. Figures showing a comparison between internal-combustion diesel-engine generation and steam generation are given later (*Fig. 2*, p. 11).

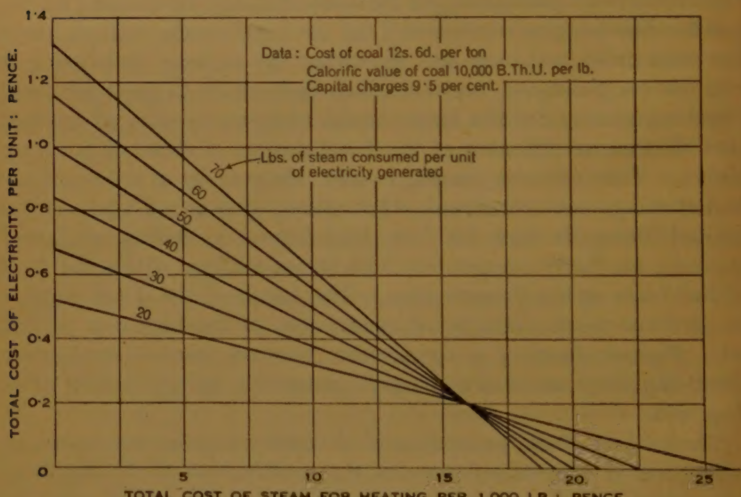
The subject of the generation of electricity from steam cannot be fully covered without reference to the combination of the supply of electricity with that of heat in the form of steam or hot water for industrial or domestic purposes. This combination has not received the attention it would appear to merit in Great Britain, because of two factors, one, the low cost of coal fuel, and the other, the equable climate.

The steam turbine lends itself specially to the abstraction of steam for heating, and as either electricity or heat can be considered as a by-product, the practice is attractive, because if the plant can be designed to give one product at a remunerative price, the other product becomes almost a gift, and the price at which it can be sold offers considerable attraction for the user. An interesting feature of such schemes is the higher overall thermal efficiency that can be obtained: this efficiency varies with the proportion of heat to electricity sent out. The greater the supply of heat in proportion

to electricity the greater the efficiency. The best annual thermal efficiency in 1936 for any selected generating station is, as already quoted, that of Battersea Power-Station, namely 27·63 per cent., and this may be compared with that of an actual industrial thermal-electric station supplying 36 lb. of steam for heating for every kilowatt-hour of electricity sent out, where the annual thermal efficiency is 65 per cent.

It is interesting to examine the cost of generating heat and electricity for various ratios of electricity to heat, and it is found that, over a wide range, the costs at the present time work out to

Fig. 1.



THERMAL-ELECTRIC GENERATION; TOTAL COST OF ELECTRICITY AND HEAT.

about 16*d.* per 1,000 lb. of process steam and to 0·2*d.* per unit of electricity. These costs are only obtained if the combined plant is designed to suit particular ratios of electricity to heat, although some allowance is made to cover a margin in the ratios. *Fig. 1* shows by separate lines the varying relative costs of steam and electricity for different ratios of steam to electricity. It will be seen, for instance, that if 18*d.* per 1,000 lb. be taken as a remunerative standard price for the supply of steam heat, the cost of the supply of electricity, with a ratio of, say, 30 to 1 of steam to electricity, will be only 0·14*d.* per unit.

In the comparison made in the previous paragraphs between

different methods of generation, no mention has been made of water-power. Great advances have been made with this form of generation, although it is desirable to state that the efficiency of the hydro-electric generating plant has much less effect on the ultimate cost of electricity than with other forms of generation. Figures showing a comparison between water-power and steam generation are given later (*Fig. 2*).

The final report of the Water Power Resources Committee, published in 1921, showed that for Great Britain, leaving out of account the question of availability, the water-power schemes then in the Committee's possession covered potential power of a total continuous capacity in excess of 250,000 kilowatts, of which Scotland had 78 per cent., Wales 14 per cent., and England only 8 per cent.

It is of interest to quote the latest available returns of the Electricity Commissioners showing the proportion of electrical energy generated by the statutory electricity undertakings in this country from the different sources available. The developed water-power, excluding private and industrial plants, is now 242,555 kilowatts, and represents 2·99 per cent. of the total plant-capacity. This is still far short of the total continuous capacity stated above, bearing in mind the relatively low average load-factor at which it works.

Type of plant.	Aggregate capacity installed : kilowatts.	Percentage of total.
Steam turbine and engine plant	7,766,752	95·89
Water-power plant	242,555	2·99
Oil-engine ,,	83,018	1·02
Gas-engine ,,	7,545	0·10
Total	8,099,870	100·00

NOTE.—This Table does not cover water-power stations used for industrial purposes.

I think I am right in stating that the lowest cost per kilowatt installed of any water-power scheme in Great Britain is that of Kinlochleven, built by the British Aluminium Company about 1907. In Papers[1] read before The Institution on those works by Messrs.

[1] A. H. Roberts, "The Loch Leven Water-Power Works." Minutes of Proceedings Inst. C.E., vol. clxxxvii (1911–12, Part 1), p. 28. F. B. Sonnenschein, "The Hydro-Electric Plant in the British Aluminium Company's Factory at Kinlochleven." *Ibid.*, p. 74.

A. H. Roberts and F. B. Sonnenschein in 1911, some figures were given which show that the capital cost of the whole scheme, excluding the aluminium factory, amounted to £28·5 per kilowatt installed, which is well within the limit for competition with steam, referred to later.

Water-power, however, can be used economically in other ways, such as by utilizing it for peak-load purposes only, either daily or annually, and consequently incurring a lower total capital expenditure per kilowatt of installed plant. Another important development of such peak-load hydro-electric stations is the use of a natural reservoir below the hydro-electric power-station from which water is pumped to the high-level reservoir during the off-peak periods. This improves the economy of operation of the steam stations working at lower load-factors and connected to the same system, and such a combination can in certain circumstances enable the system as a whole to secure a considerable reduction in total generating costs. A number of such peak-load water-power and pumping stations have been installed in Europe, but so far none has been built in Great Britain. There is undoubtedly a possibility, in the future, of such schemes in this country resulting in economy, especially if the price of fuel continues to rise.

It is of considerable importance to bear in mind that the principal items in the cost of generation which largely influence the resultant cost of electricity, are the cost of the fuel in a steam station, and the capital cost of the civil engineering works with the rate of interest on such capital cost in a hydro-electric station.

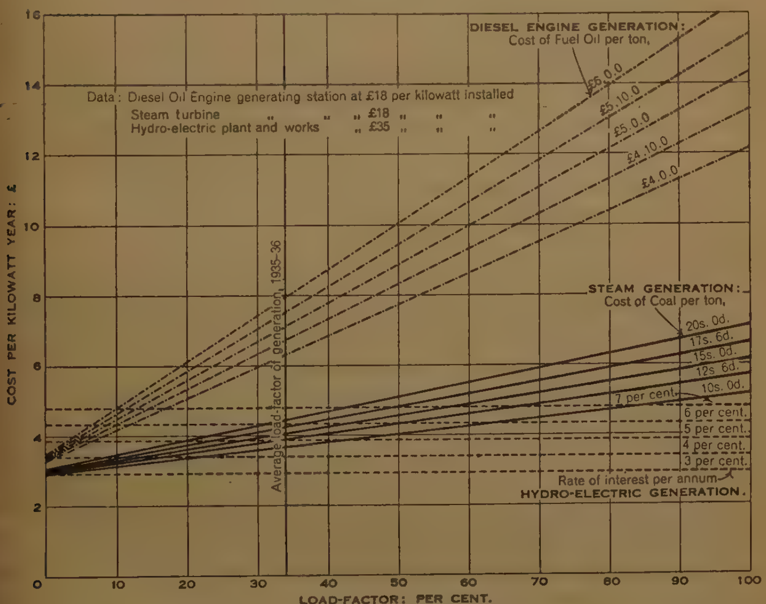
Putting on one side all political aspects, and considering the matter on a purely hypothetical basis without reference to other possibly unknown factors, it may be of interest to show, as in *Fig. 2*, the variation in the total cost of electricity generated per kilowatt year, in steam stations and in hydro-electric stations, when plotted to a base of varying load-factor. The set of full lines shows the results for steam generation with coal at varying prices, whilst the set of dotted lines shows the results for hydro-electric generation with varying rates of interest. The curves are based on the cost of hydro-electric works taken at an average figure of £35 per kilowatt installed, and on that of steam stations at an average figure of £18 per kilowatt installed, with a rate of interest on capital taken at 5 per cent. The relation of the curves shows hypothetically where one system is more economic than the other.

Fig. 2 also shows, by a set of dash-dot lines, the cost of electricity generated per kilowatt year for diesel-engine stations with oil fuel at varying prices. This indicates how the high cost of fuel oil affects

the results and how such stations can only be economic in large sizes when the load-factor is very low.

Taking an example to explain one use of the figure, we see that water power at £35 per kilowatt installed with interest at 5 per cent. per annum is more economic than steam power costing £18 per kilowatt installed with coal at 10*s.* a ton if the load-factor of operation exceeds 45 per cent. Another example shows that water power at £35 per kilowatt installed and with interest again at 5 per cent.

Fig. 2.



COMPARATIVE TOTAL COSTS OF GENERATION OF ELECTRICITY.

is more economic than diesel oil-engine power costing £18 per kilowatt installed, with fuel oil at £4 a ton if the load-factor of operation exceeds 7 per cent. It can also be inferred from *Fig. 2* that water power can be economically used for peak-load purposes, say at 10 per cent. load-factor, when the total cost of its works does not exceed £30 per kilowatt installed.

Before leaving the subject of the generation of electricity it is of importance to mention the gain in economy that has resulted from

the passing of the 1926 Electricity Act and the operation of the Central Electricity Board's system. For the sake of those who are not conversant with the electrical side of engineering, it should be explained that the network of transmission lines, known as the "Grid," forms the connexion between the selected generating stations and provides for the transfer of load between such stations and the other centres of distribution which at one time had generating stations but whose stations have been shut down as being uneconomic. The advantage gained is, therefore, due not only to the operation of the more economic stations but to the very great saving in the installation and purchase of standby plant. In 1926, before the Act came into operation, the percentage of standby plant to total plant installed was of the order of 70 per cent. In consequence of the operation of the Board this percentage was only 34 per cent. in 1935–36. These two factors together have done a great deal in reducing the cost of electricity to the Undertaker who has to carry out its distribution. If the cost of electricity to the consumer is now very variable and in many cases still far too high for it to be a real commodity, the cost of distribution is more at fault than the cost of generation.

The combined economic operation of the selected generating stations in Great Britain, interconnected by the Central Electricity Board grid lines, is controlled by the Central Electricity Board from six control-rooms for the whole of England and Wales and from one for the whole of Scotland. In each area it is the duty of the staff operating the system to scheme out for each week in the year and for each day of the week the most economic combination of selected stations to provide for the forecasted load-curve of those periods. The control-room staff are in telephonic communication with every generating station and can give orders to the staff at these stations as to how much load should be taken up or taken off. If a fault occurs at any generating station or substation or on any transmission-line between stations, the fault is automatically cut out and is indicated in the control-room by instruments and on a diagrammatic model of the electrical connexions in the area which simulates the condition of switches therein.

Generating stations connected to the grid run at constant frequency within extremely small limits, such limits being required to allow the transfer of load in one direction or the other. This control of frequency is so successful that the variation in frequency over one of the Board's largest areas, which includes London, is such that the difference between astronomic time and electrical time measured at 50 periods per second never exceeds ± 5 seconds

at any moment, and the mean variation over 24 hours does not exceed ± 1 second.[1]

Distribution of Electricity.

It is necessary, as a first step in referring to schemes for the distribution of electricity, to leave out of the problem the extra-high-tension transmission-lines whose duty it is to form part of the whole system of economic generation. Distribution, therefore, starts at the bus-bars of generating stations or substations where electricity at extra high voltage is transformed to a reasonable economic voltage for the primary distribution.

The capital and working costs of the distribution-system of any undertaking are more than half the total cost of the whole undertaking. During the period 1924 to 1933 the average total distribution costs per unit sold by Authorized Undertakings in Great Britain increased from 0·7573*d.* to 0·7664*d.* per unit, although, expressed as a percentage of distribution capital, they fell from 15·16 to 13·13 per cent. During the same period the number of units sold per £1 of distribution capital decreased from 48·04 to 41·13 units. It is satisfactory to note that since 1933 this figure of 41·13 has recovered somewhat to about 46 units sold per £1 of distribution-capital. This indicates the necessity for reducing working costs, and for distributing the largest possible number of units over a distribution-system of given capital expenditure. The lack of improvement in the cost of distribution over a considerable number of years shown by the above figures has caused the industry and the Government some concern, although it must not necessarily be argued that the large increase in rural distribution has been the reason for bringing this about. It may be shown from recent experience that the capital expenditure per consumer on distribution in certain rural areas is less than that in many urban areas, and the capital cost per consumer for the country as a whole has decreased steadily in spite of the development of the supply in rural areas.

If distribution-costs are to be reduced and the efficiency of distribution increased, it is necessary to see how improvements can be made. It has been found, for instance, that the physical shape of

[1] The President then showed, with the permission of the Central Electricity Board, a film illustrating the Battersea Power-Station, the Barking Transformer Station, the largest switching substation in Great Britain at Northfleet, and some views of transmission-lines and control-rooms.

the area of supply of some electricity undertakings is such that outlying parts thereof may be at a greater distance from the Undertakers' source of supply than from the source of supply of another neighbouring undertaking. Economically the district should have been supplied from that other Undertaker.

As a result of the many facts mentioned above, the Government held an inquiry by a Committee under the Chairmanship of Sir Harry (now Lord) McGowan, and the report of this Committee, known as "The McGowan Report," was issued in May, 1936, and dealt with the whole position as regards distribution and made recommendations to effect improvements. It also recommended methods of ensuring and expediting the standardization of systems, pressures, and methods of charge.

The recommendations of that Committee dealing with suggested changes in organization to provide for economy of distribution have created a great deal of criticism in the industry. There can hardly be any doubt in the mind of an impartial critic that the recommendations in the Report, as regards the compulsory amalgamation of the smaller undertakings with the larger to form a larger single entity, is correct in principle, and that it is only the proposed methods of carrying this out which are subject to criticism. The objects aimed at are to benefit the consumer by lowering the price of electricity, and by making electricity available at an attractive price in places where it is not yet supplied. It may be hoped, therefore, that some solution of the problem can be found.

The cost of the distribution of electricity depends fundamentally on the density of the individual loads of each consumer over a given area and the distance of such loads from the source of supply. Many years' experience proves that the lower the price of electricity to consumers the greater is the number of units taken by them.

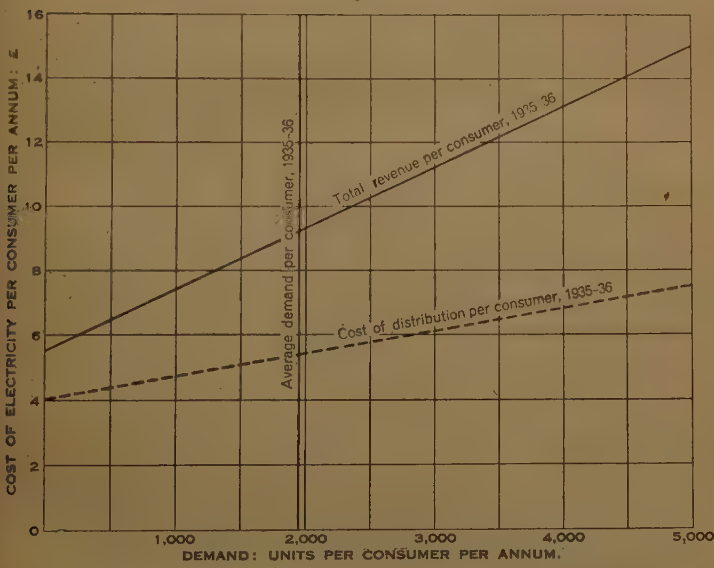
Fig. 3 has been prepared from data available for the year 1935–36 in which the average units sold per consumer per annum were 1955. The lower dotted line shows how the total annual cost of distribution, and the full line how the total revenue per consumer, both increase with the demand. It will be seen from the slope of the lines that the popularity of the use of electricity is still growing, and there is no indication that the demand for electricity by consumers has reached saturation.

Comparing the cost of primary distribution by overhead lines and by underground cables, and taking the lowest reasonable cost of both methods, the cost of underground cables is from 3 to 6 times greater than that of overhead lines, over a range of voltage from 6,600 to 33,000.

The good work done by the Electrical Research Association has been instrumental in reducing very largely the cost of the installation of underground cables, and is likely to continue to do so in the future, especially as regards higher voltages.

In certain districts and in certain climates, especially overseas, it may be inexpedient to rely alone on overhead lines for fear of damage from thunderstorms or excessive wind-velocities. In Great Britain the design of transmission-lines has been improved very greatly in the last few years from the point of view of security against storm

Fig. 3.



Average Annual Cost of Electricity per Consumer.

damage. An interesting sidelight on snow-laden lines is that more failures occur in sheltered valleys than over exposed hills. Overhead earth-wires are provided on most transmission-lines, for the prevention of damage due to lightning. The practical value of overhead earth-wires depends on the resistance to earth of the towers being so low that the tower-potential due to lightning is less than the flashover voltage of the insulators. It is therefore obvious that earth-wire protection will not be effective below a given figure of line insulation, and hence the well-known experience that the higher the voltage of any transmission-line, the less

is it likely to be adversely affected by lightning. In mountain districts with rock foundations it is often exceedingly difficult to earth each tower properly so as to provide the low resistance required, and in such cases the earth wire has to act as a conductor for lightning current to a greater extent than usual. In Great Britain, during the last 6 years, the number of faults on the Central Electricity Board's overhead system due to lightning was only 20 per cent. of that of those due to all other causes.[1]

The British Post Office authorities give valuable help to distribution-engineers in locating incipient faults on transmission-lines, and a number of ingenious instruments have been devised for locating defects in insulation, etc. An interesting development has recently been tried to enable faulty line-insulation to be detected, before it has completely broken down, by means of a portable wireless-set.

Overhead transmission suffers from the difficulty which arises from what local antagonists call its intrinsic ugliness, more particularly in rural districts where communities depend largely for their support and livelihood on visitors who come to enjoy the scenery. The question for any community to decide is whether the extra comfort, convenience, and saving of labour that electricity gives is worth more to them than any loss suffered as a consequence of the effect of the appearance of the lines on the inhabitants or on visitors.

A balance has to be created between cause and effect which is not easy to the engineer, however beautiful he may think his design is for the transmission-line. A pole, or tower, and a line may be a beautiful piece of engineering design just as a bridge or a gasometer may be. The engineer, however, cannot get the public to admit that what may be a beautiful design to him is a thing of beauty to them. Ruskin's theory that an object of perfect utility without redundant material must be a thing of beauty is not always upheld by the modern critic, but the present younger generation at least has the advantage over the preceding one in that, in regard to modern engineering structures of all kinds, it grows up to the newer forms of structures and does not particularly notice them.

Electricity as a Commodity.

The electric-supply industry has been often blamed for con-

[1] As an example of the speed of erection of the Central Electricity Board's transmission-line towers, the President showed a film of the last tower to be put up to complete the Central Electricity Board's system in September, 1933, the site being near Fordingbridge in Hampshire. He mentioned that the time taken to complete the erection of the tower was 75 minutes.

tinuing to charge excessive prices for electricity. A famous Belgian company, the *Société Financière de Transport et d'Entreprise Industrielle*, in its annual report for 1936 shows by comparing the cost of various domestic commodities throughout Europe, ranging from electricity to coal and including butter, bread, milk, meat, potatoes and sugar, that the price of electricity has fallen more since 1914 than that of any other commodity, whilst the price of electricity in Great Britain has fallen more than in any other country. That it can fall still farther is practically certain, as I have mentioned earlier in this Address.

The consumer of electricity is demanding better service, an equality of tariffs for similar classes of users, standardization of voltage, and availability of supply in rural areas. These matters have already been referred to. The greatly increased use of electricity and especially of alternating current has been the cause of showing up weaknesses in the wiring installations on consumers' premises. New installation-work in private consumers' premises is often carried out badly, does not comply with voluntary rules and regulations, and by some means gets connected, despite the statutory regulations binding on the supply authority before it gives a supply to new premises. Such badly-designed and/or badly-constructed wiring installations have been the cause of an undue number of fatal accidents to users, most of which need not have happened. The number of fatal accidents in Great Britain over the last 5 years is shown on *Fig. 4* (p. 18) and it will be seen that the mean rate of increase is considerable. It should also be noted that the mean rate of increase is greater in domestic than in factory premises. Factory installations are subject to Home Office regulations, whereas domestic premises are not.

The number of fires caused by faulty installation-work is also far too great. Fires proved to be due to electrical causes in premises of all kinds in the year 1936 were of the order of one per 2,000 consumers for the whole country.

Such defects and the resulting damage, injury and casualties hinder progress, and to improve the position, especially in regard to domestic premises, several branches of the electrical industry are, at the suggestion of the Association of Supervising Electrical Engineers, now trying to formulate means by which wiring-installation contractors and operatives shall be registered, and, to be so, must possess a required standard of knowledge and craftsmanship. They also propose to make compliance with the present voluntary wiring rules and regulations compulsory, and to see that only fittings and apparatus of defined standards of quality are permitted to be used.

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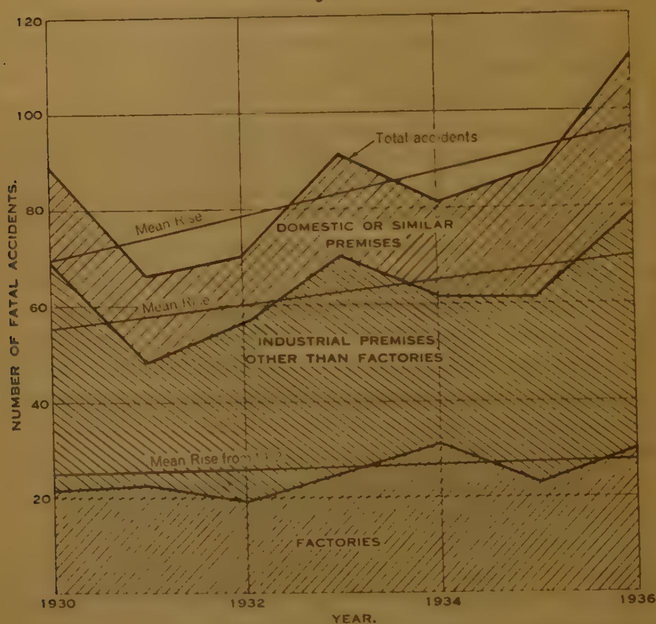
This development, which is intended primarily to safeguard the user and to make electricity more popular, deserves encouragement.

I have given you a heterogeneous collection of facts and prospects, and apologize if they appear to circumnavigate applied science. Perhaps the sentiments of my late chief, Sir Alexander Kennedy, are appropriate in the circumstances and give the necessary excuse for one of my age : " As years go on and experience ripens and accumulates, one's work has to deal more and more with men and matters,

Fig. 4.



Fatal Electrical Accidents.

with general schemes and methods, with financial possibilities and balance sheets. It deals too little with directly mechanical or electrical problems which fascinated us when we were younger and for the sake of which probably we took to engineering at all in the first instance." That is one of the penalties of engineering old age, and there is no use lamenting it.

We have mostly to leave the detailed mechanical problems mentioned to the younger men, and our compensation is their obvious enjoyment in the solution of them. We can at least advise them on the relative

merits of their schemes as the result of our longer experience, and so perhaps save them time and sometimes fruitless labour.

Sir Alfred Ewing said, in the James Forrest Lecture of 1928[1]: "All our efforts to apply the sources of power in nature to the use and convenience of man, successful as they are in creating for him new capacities, new comforts, new habits, leave him, at bottom, much as he was before." Civilization turned the weapons provided by science upon herself, and he pointed out most aptly that "progress in the arts that we engineers have learnt has far outstripped the ethical progress of the race." The Great War, in fact, indicated a moral failure of the use of applied mechanics, and he compared the condition of affairs in it to giving a child a sharp-edged tool before he had the sense to handle it wisely.

Mr. H. G. Wells, in discussing the question of recurring confusion between opposing sets of principles in relation especially to foreign affairs, said that an enormous majority of people in the present world would vote for perpetual peace and an absolute end of war for ever if they were given the chance, but if they were asked to accept the broad things that world peace certainly entails, they would not have the least idea of what that really implied.

Ewing's moral philosophy and Wells' opinion of the human race suggest the necessity for a greater advance in the study of political responsibility in order to keep pace with the advance in the study of applied science. I think, therefore, that I cannot do better, in concluding my Address, than refer to a speech by Lord Weir, when opening recently a new Research Laboratory, in which he stated: "I should like to see some millions spent every year in research in the political sphere for the founding of a real thinking department for the analysis of the investigation of human qualities. Can we not explore the possibility of healing and softening racial bitterness; can we not attempt to find some method of dissolving national resentments and ultimately demonstrate and make clear to the world the utter inability of war to solve any political problems?"

Sir Cyril Kirkpatrick, Past-President, moved:—

> "That the best thanks of The Institution be accorded to the President for his Address, and that he be asked to permit it to be printed in the Journal of The Institution."

In doing so, he remarked that those present were all very much

[1] "A Century of Inventions." Minutes of Proceedings Inst. C.E., vol. 226 (1927–28, Part II), p. 387.

wiser for having heard the President's Address. To his mind, it had been a fascinating Address, full of interest not merely to the purely "electrical," "mechanical," or "civil" engineer, but in a wonderful way to The Institution of Civil Engineers as a whole, embracing as it did all branches of the engineering profession.

Mr. C. L. Howard Humphreys, in seconding the motion, reminded the President of a conversation which they had had some time previously, when the President had been considering the form which his Address should take. He had then ventured to suggest that the President should give an account of the development of modern electrical engineering as it affected the ordinary person, but the President had remarked, "Oh, they know all about that!" Mr. Humphreys, however, was glad that the President had changed his view, for his audience had spent a most interesting evening, and had learned a very great amount from the Address. He felt sure that all those present had greatly appreciated it.

The resolution was carried by acclamation.

The President thanked the proposer and seconder for their kind remarks, and those present at the meeting for carrying the motion. He was extremely grateful for the manner in which his Address had been received, and he would be proud indeed to see it printed in the Journal of The Institution.

MEDALS AND PREMIUMS.

The PRESIDENT presented the Telford Gold Medal, the James Watt Gold Medal, the Coopers' Hill War Memorial Prize, the James Forrest Medal, and the Howard Quinquennial Prize, and the awards for Session 1936–37 were announced, as follows :—

FOR PAPERS READ AND DISCUSSED AT THE ORDINARY MEETINGS.

The following, being Members of Council, are ineligible to receive awards for their Papers, and the Council have expressed to them the thanks of The Institution :—

Raymond Carpmael, O.B.E., for his Paper on "The Maintenance of Waterways to Harbours and Docks."

Frederick Charles Cook, C.B., D.S.O., M.C., for his Paper on "Road Design and Road Safety."

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1. A Telford Gold Medal to David Mowat Watson, B.Sc., M. Inst. C.E., for his Paper on "West Middlesex Main Drainage."
2. A James Watt Gold Medal to Sir Noel Ashbridge, B.Sc., M. Inst. C.E., for his Paper on "Modern Developments in Broadcasting Transmission and Television."
3. The Coopers' Hill War Memorial Prize for 1936–37 to John Guthrie Brown,[1] M. Inst. C.E., for his Paper on "Kincardine-on-Forth Bridge."
4. A Telford Premium to Arthur Holden Naylor, M.Sc., B.Sc. (Eng.), M. Inst. C.E., for his Paper on "The Second-Stage Development of the Lochaber Water-Power Scheme."
5. A Telford Premium to Stanley Fabes Dorey, D.Sc., M. Inst. C.E., for his Paper on "Welded Joints in Pressure-Vessels."
6. A Telford Premium to Frederick William Adolph Handman,[2] C.B.E., M. Inst. C.E., for his Paper on "The Lower Zambezi Bridge."
7. A Telford Premium to Cecil Lee Howard Humphreys, T.D., M. Inst. C.E., for his Paper on "The Reconstruction of the Chester–Holyhead Road, near Penmaenmawr, North Wales."
8. A Telford Premium to George Eric Howorth, M.C., B.Sc., M. Inst. C.E., for his Paper on "The Construction of the Lower Zambezi Bridge."

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[1] Has previously received a Miller Prize.

[2] Has previously received a Telford Premium.

9. A Telford Premium to Benjamin Williams Huntsman, B.Sc., (Eng.) M. Inst. C.E., for his Paper on "The Salonika Plain Reclamation-Works."

10. A Telford Premium to Ludolph Reinier Wentholt, D.Tech.Sc., M. Inst. C.E., for his Paper on "Ship-Canals Utilized for Drainage."

11. A Telford Premium to Professor Stephen Mitchel Dixon,[1] O.B.E., M.A., B.A.I., M. Inst. C.E., Gerald FitzGibbon, B.A., B.A.I., and Michael Anthony Hogan, D.Sc., Ph.D., M. Inst. C.E., jointly, for their Paper on "The Flow of the River Severn, 1921–1936."

12. A Telford Premium to Professor Alfred John Sutton Pippard,[1] M.B.E., D.Sc., M. Inst. C.E., Eric Tranter, B.Sc., Stud. Inst. C.E., and Letitia Chitty, M.A., jointly, for their Paper on "The Mechanics of the Voussoir Arch."

13. A Telford Premium to Alec James Dean, B.Sc. (Eng.), Assoc. M. Inst. C.E., for his Paper on "The Lake Copais, Boeotia, Greece: Its Drainage and Development."

14. An Indian Premium to Herbert John Nichols, D.Sc., M. Inst. C.E., for his Paper on "Pre-Stressing Bridge Girders."

15. A Trevithick Premium to Donald Arnott Stewart, for his Paper on "Fundamental Research on the Application of Vibration to the Pre-Casting of Concrete."

For Papers Published with Written Discussion.

1. A Webb Prize to Paul Lewis Henderson, Ph.D., M.E., Assoc. M. Inst. C.E., for his Paper on "Some Developments in Railway-Carriage and Wagon Construction."

2. A Manby Premium to Professor Herbert Walker Swift, M.A., D.Sc. (Eng.), for his Paper on "Fluctuating Loads in Sleeve Bearings."

3. A Crampton Prize to Bertram Darell Richards, B.Sc. (Eng.), M. Inst. C.E., for his Paper on "Flood-Hydrographs."

4. A Telford Premium to William Henry Glanville,[1] D.Sc., Ph.D., M. Inst. C.E., and Frederick George Thomas, B.Sc., Assoc. M. Inst. C.E., jointly, for their Paper on "The Redistribution of Moments in Reinforced-Concrete Beams and Frames."

5. A Telford Premium to John Christian Richards, B.E., B.A., for his Paper on "Stress-Determination for a Three-Dimensional

[1] Has previously received a Telford Premium and a Miller Prize.

Rigid-Jointed Framework by the Method of Systematic Relaxation of Constraints."

6. A Telford Premium to Robert Ferguson, B.A., B.E., M. Inst. C.E., for his Paper on "Newry Ship-Canal Improvement Scheme."

7. A Trevithick Premium to Eric Norman Webb, D.S.O., M.C., M. Inst. C.E., for his Paper on "Efficiency Tests of Large Modern Pelton Wheels."

8. A Telford Premium to Professor Brian Laidlaw Goodlet, M.A., Assoc. M. Inst. C.E., for his Paper on "The Impedance of Transformers Connected in Cascade."

For Papers Read at Students' Meetings in London and by Students before Meetings of Local Associations.

1. The James Forrest Medal and a Miller Prize to Dennis Frank Orchard,[1] B.Sc. (Eng.), Stud. Inst. C.E., for his Paper on "A Survey of the Present Position in Road Transition-Curve Theory."

2. A Miller Prize to Sydney Kenneth Jordan, Stud. Inst. C.E., for his Paper on "Foundations for Basement Buildings adjoining Existing Property."

3. A Miller Prize to Nicholas Charles Callard de Jong, B.Sc. (Eng.), Stud. Inst. C.E., for his Paper on "Telephone Cables."

4. A Miller Prize to Donald Mackenzie Hamilton, B.Sc., M.Eng., Assoc. M. Inst. C.E., for his Paper on "Some Civil Engineering Works in America."

5. A Miller Prize to Richard Cecil Whitehead, B.Sc., Stud. Inst. C.E., for his Paper on "Purification of Public Water Supplies."

6. A Miller Prize to Henry Grace, B.Sc., Stud. Inst. C.E., for his Paper on "Bridgwater Corporation Water-Supply, Taunton and Bridgwater Canal and R. Parratt Crossings."

7. A Miller Prize to Francis Colin Squire, B.Sc. (Eng.), Stud. Inst. C.E., for his Paper on "Rainfall."

Baker Gold Medal.

The Baker Gold Medal for the triennial period 1934–37 awarded to Bo Manne Hellstrom,[2] M. Inst. C.E., for his Paper on "The Perak River Hydro-Electric Power Scheme."

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[1] Has previously received a Miller Prize.
[2] Has previously received a Telford Gold Medal.

Howard Quinquennial Prize.

The Howard Quinquennial Prize for the quinquennial period 1932–37 awarded to Professor John Fleetwood Baker,[1] M.A., D.Sc., Assoc. M. Inst. C.E., for his Paper on " The Rational Design of Steel Building Frames."

Bayliss Prizes.

Bayliss Prizes awarded on the results of the October, 1936, and April, 1937, Examinations, respectively, to John Macdonald Gordon Forsyth, B.Sc. (Eng.) and Shanti Swarup Varma, Stud. Inst. C.E.

Charles Hawksley Prize.

A Charles Hawksley Prize of £150 for 1937 to Frank Robert Bullen, B.Sc., Assoc. M. Inst. C.E.; Edwin Lomax, Assoc. M. Inst. C.E., and Andrew Cairns Ross, B.Sc., Stud. Inst. C.E., received " Honourable Mention " and grants of £50 each.

[1] Has previously received a Telford Gold Medal and a Telford Premium.

Paper No. 5052.

"A Pre-Cast Reinforced-Concrete Underline Railway-Bridge."

By Henry George Follenfant, B.Sc. (Eng.), Assoc. M. Inst.C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

Introduction.

This Paper deals with the design, construction, and erection of a reinforced-concrete underline railway-bridge carrying the Piccadilly line of the London Passenger Transport Board over the ticket-hall of the reconstructed South Harrow station. The work was completed in the autumn of 1934. The noteworthy feature of this bridge is that it was constructed complete in a temporary position adjacent to the tracks, and during a Sunday possession of the line it was rolled into position.

Previous to the work of reconstruction, South Harrow station consisted of two solid-filled platforms partly covered with timber roofs, with booking-office and entrance in South Hill Avenue, some 160 yards from Northolt Road, the main thoroughfare. The work of reconstruction included lengthening the platforms by approximately 200 feet, and the building of station entrances and a ticket-hall adjacent to Northolt Road. This necessitated the bridging over the area of the new ticket-hall for a distance of some 50 feet back from the abutment of the existing underline bridge over Northolt Road. The bridge carrying the tracks over the new ticket-hall is the subject of this Paper. The site of the bridge and the relationship of the old and reconstructed station is shown in *Fig. 1* (p. 26), and the outline-sections of the bridge are shown in Figs. 2, Plate 1.

Main Considerations affecting Design.

The main considerations affecting the design were as follows :—

(1) Every precaution should be taken to ensure that the bridge

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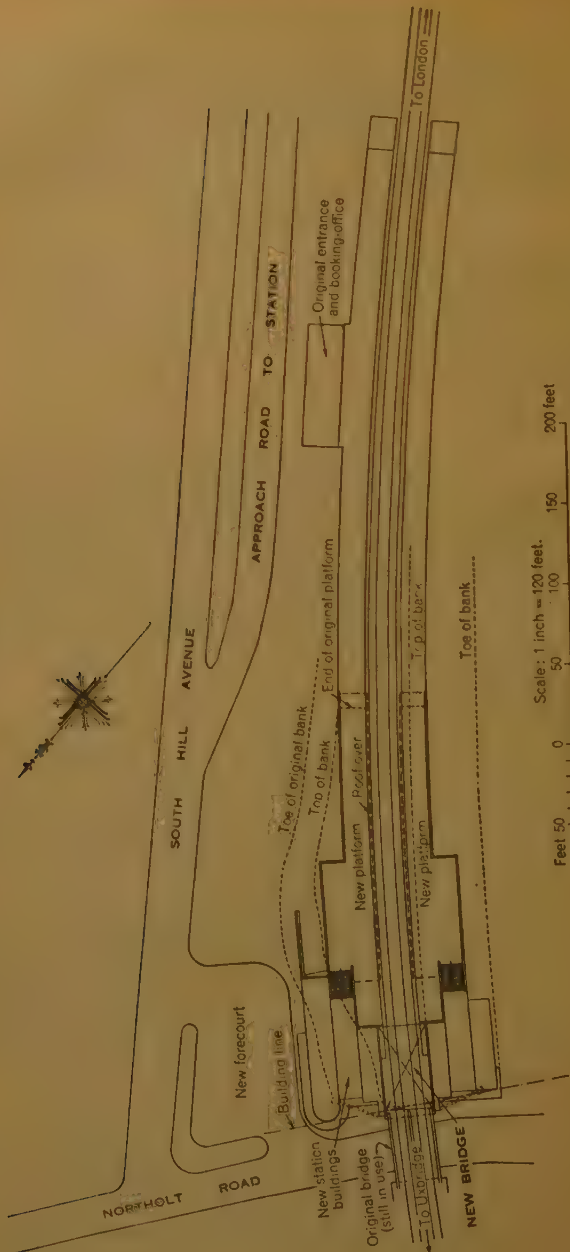
[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—Acting Sec. Inst. C.E.

Fig. 1.



Plan of Station and Site of New Bridge.

should be waterproof. This was especially important as the soffit of the bridge formed the ceiling of the ticket-hall.

(2) That there should be the minimum of interference with the tracks during erection.

(3) The bridge should be a structure free to deflect independently of the station-building roof.

(4) Provision should be made for a future widening of Northolt Road and the opening-up of the space now occupied by the abutment to the bridge over Northolt Road as a central entrance to the ticket-hall. For this reason the new bridge consists of two spans, one of 41 feet 9 inches at the east end and one of 9 feet 6 inches at the west end, the latter being a temporary provision until the widening of the road should be taken in hand. The west end of the 41-foot 9-inch span is supported on two columns, with a free space of 17 feet between them for a future central entrance.

The east end of the longer span is carried on a mass-concrete abutment-wall and the west end of the short span is carried on new brickwork built up behind the existing brick abutment to the Northolt Road bridge.

Alternative Designs.

Among the designs of bridge examined were :—

(1) A span of three steel girders with joist flooring, the steel being inserted under the tracks as far as possible during the day. The track would be supported by rail-bearers on timbers spaced to miss the cross-joists.

(2) A similar arrangement with the transverse joists encased in concrete and the whole built up in a temporary position and rolled into its permanent position.

(3) A bridge consisting of five longitudinal plate-girders encased in concrete and braced transversely with reinforced-concrete top and ceiling slabs; the whole to be built up in a temporary position and rolled into the permanent position.

The construction of the longer span complete in a temporary position, and rolling-in, was considered to be the method of erection which would cause least interference with the permanent way and was most likely to ensure the bridge being waterproof.

The reinforced-concrete design which was finally adopted was considered to meet the primary requirements and to have the following advantages over the steel designs :—

(1) Most of the steel would have had to be encased in concrete to meet the architectural requirements or for protection, as the steel-work would be mostly inaccessible for painting after the completion

of the station-building roof. The concrete encasing is wasteful, increasing the dead load and not contributing to the strength.

(2) It was estimated to save 20 per cent. of the cost of the most suitable steel design.

(3) There would be a saving in weight to be rolled into position.

(4) The best type of finish to the ticket-hall ceiling was likely to be obtained ; for example, the lines of steel flanges encased in concrete often show as slightly differently-coloured concrete.

Design.

The calculations used in the design of the reinforced-concrete bridge do not present any unusual features. Working stresses of 16,000 lbs. per square inch tension in steel and 600 lbs. per square inch compression in concrete, with a modular ratio of 15, were used. These stresses are, perhaps, low for modern design, but in view of the possible accidental stresses in the process of rolling-in and to cover fluctuating stresses and live-load shocks, a higher " factor of ignorance " was considered desirable. The equivalent uniformly-distributed loading from the Report of the Bridge Stress Committee was used.

The top slab and main longitudinal beams of the 41-foot 9-inch span are designed as T-beams from 3 feet 11 inches to 4 feet 2 inches overall depth, six of which have a 1-foot 6-inch wide rib, whilst the outside beams have a 9-inch wide rib. The top slab is 27 feet 6 inches wide and varies in thickness from 8 inches in the centre of the span to 5 inches at the ends, thus forming a camber for drainage and an increase in effective depth in the T-beams at the centre of the span. To provide lateral stiffness and to equalize the deflexion amongst the longitudinal beams five transverse beams are provided. Manholes with 2-foot by 1-foot 6-inch clear openings are arranged opposite the spaces between the longitudinal beams to allow the internal shuttering to be removed, to inspect the internal faces and to permit the installation of electric-light wiring. The soffit of the bridge is the ceiling of the ticket-hall, and 3-inch ceiling-slabs are provided between the beams. At the west end the main beams are carried by a bolster beam supported by columns at 20-foot 6-inch centres, and cantilevering over the columns for a distance of 2 feet 10½ inches at each end. The additional depth required for this beam is provided by the ticket-hall ceiling being at a lower level for a short distance at the west end. To avoid abrasion between the two concrete surfaces, and to obtain as far as possible an even bearing surface, a ¼-inch thickness of sheet lead is laid on the tops of the columns. For the rolling-in it was necessary to provide an end transverse beam carrying the longitudinal beams at the mass-concrete abutment end, and in the permanent position a continuous bearing is

obtained by a 100-lb. flat-bottom rail upside down cast in this beam. The head of the flat-bottom rail bears on the web of a 9-inch by 4-inch rolled-steel joist on its side in the top of the abutment, the bearing surfaces being separated by a $\frac{1}{4}$-inch lead sheet to obtain a continuous even bearing.

The top slab is waterproofed by a $1\frac{1}{8}$-inch thickness of a mixture of natural rock asphalt, Trinidad bitumen, and graded grit laid in three layers while the bridge was in the temporary position. Flexible waterproof joints at the abutment and over the joints between the two spans are made with No. 16 S.W.G. copper sheeting fastened to the concrete with copper bolts and asphalted over. The asphalt is protected with a 3-inch slab of concrete over the whole surface.

The 9-foot 6-inch span consists of two 15-inch deep slabs 13 feet 9 inches wide, waterproofed in the same way.

Construction.

The concrete was made with 3 parts by volume of $\frac{3}{4}$-inch natural Thames shingle to $1\frac{1}{2}$ parts of sand and 1 part of Portland cement. Test-cubes showed a crushing strength of from 3,800 to 4,000 lbs. per square inch after 28 days.

The 41-foot 9-inch span was cast on a temporary timber staging in a position on the north side of the line exactly parallel with, and 41 feet away from, its permanent position. The embankment was excavated for the erection of the staging, which consisted of four trestles in a direction at right angles to the span, built on concrete footings on good clay. The trestles were spanned by 12-inch by 12-inch timber beams, which were spaced under each main longitudinal beam of the bridge. $1\frac{1}{4}$-inch decking boards were supported on 9-inch by 3-inch timber joists and folding wedges on the timber beams. The whole of the soffit-shuttering was lined with "Masonite" to give a smooth surface to the ceiling. Figs. 3, Plate 1, show the arrangement and details of the temporary staging.

The 3-inch ceiling-slabs were first cast with their reinforcement running through the beam-positions. The shuttering for the beams was then erected on the ceiling-slabs as a series of hollow boxes. The casting of the 41-foot 9-inch span was carried out in one operation in 14 hours, approximately 100 cubic yards of concrete being poured in that time, using one $\frac{1}{2}$-cubic-yard and one $\frac{1}{4}$-cubic-yard mixer. The concreting was interrupted only for $1\frac{1}{2}$ hour for the placing of the top-slab steel.

The 9-foot 6-inch span slabs were cast in a position on the embankment to the east of the abutment as close to the tracks as possible, so that they could be lifted by the railway steam-crane.

For the rolling-in, the larger span was cast over four carriages made up of two pieces of 80-lbs. bull-head rail on edge 2 feet 3 inches long, bolted to a 1¼-inch plate which was clipped around the main beams with angle-cleats. The carriage rode over fourteen 3¼-inch diameter balls. The arrangement of the rolling-in carriage is shown in *Figs. 4*. The rolling-in tracks consisted of a pair of 80-lb. rails on edge, tack-welded to a ½-inch bed-plate. At the east end the tracks were carried on the mass-concrete abutment 1 foot 6¾ inches from the centre-line of the abutment-beam, and at the west end they were carried on a temporary timber trestle built normal to the direction of the span and 2 feet 7½ inches from the centre-line of the bolster-beam. The arrangement of the tracks in relation to the construction staging is shown in Figs. 3, Plate 1.

Two drawbars of 6-inch by ½-inch plate were cast in the ends of the two outer transverse beams; they extended to the centre of the first 1-foot 6-inch main beam from the south side and had a piece of 1-inch diameter bar at right angles as an anchor. The drawbars were connected by three- and two-sheave blocks and falls to two hand-operated winches situated on the embankment at the south side of the bridge position; the winches were each weighted down with 6 tons of scrap rail.

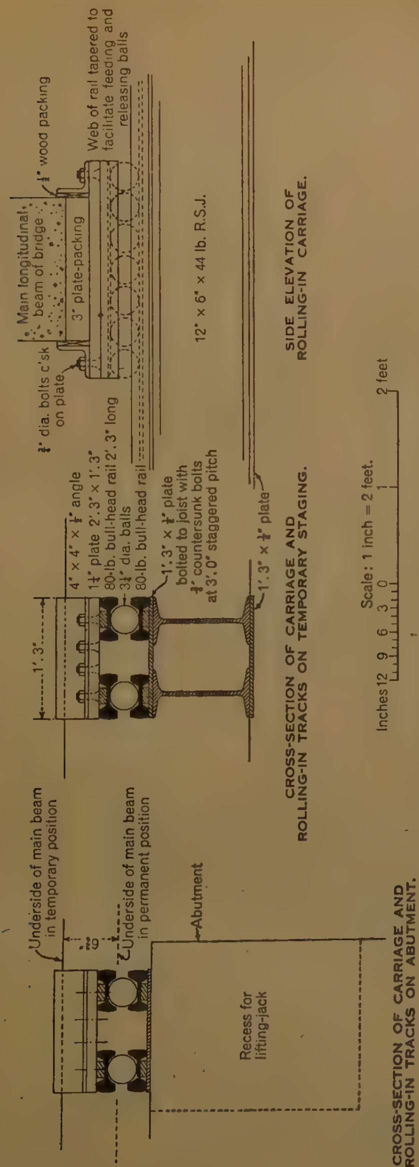
The permanent way was supported temporarily on 24-inch by 7½-inch rolled-steel joists and timber sills to enable the abutment and columns to be built in trench, and for the space to be excavated for the bridge.

Erection.

For the erection, the engineers had complete possession of the line from the last train on Saturday night, 15 September, 1934, until the first train on Monday morning, 17 September. The bridge was hauled as close as possible to the edge of the tracks during the previous week, and about 30 cubic yards of permanent-way slag were spread over the surface. On the morning of the 16th September the permanent way and temporary supports were removed by the Board's permanent-way staff with the assistance of two steam cranes. The complete track from the westbound road was transferred to the new bridge by the steam crane standing on the eastbound road.

The 41-foot 9-inch span was then hauled into position at the rate of 12 inches a minute. The rolling-in tracks were marked at 3-inch intervals and the relative travel of the ends of the span were checked by plumb-bobs hanging over the tracks. The sideways positioning was obtained accurately and easily. It is interesting to note that

Figs. 4.



ARRANGEMENT OF ROLLING-IN CARRIAGE

the pull was done almost entirely by one winch, and the work done on the other winch was just sufficient to keep the bond tight. Some trouble was caused by the bridge arriving $\frac{5}{8}$ inch west of its proper

position, due apparently to the ball-bearings travelling to the west side of the webs of the rolling-in rails, the small clearance between the balls and the web allowing this to occur. It was intended that after hauling into position the bridge should be lifted at one end by two 100-ton hydraulic jacks operated from one pump, and the two carriages removed. The procedure was then to be repeated at the other end. At the east end the jacks were situated in pockets left in the top of the abutment with the rams projecting through the carriages. At the west end the jacks were situated on the concrete bases of the columns and exerted their pressure through steel columns.

Owing to the bridge getting out of position, however, plate-packing and rod-rollers were placed at the four lifting points, and the bridge was pushed into its correct position by jacking longitudinally off the abutment wall at the west end. The west end was then lowered on to the columns, followed by the east end.

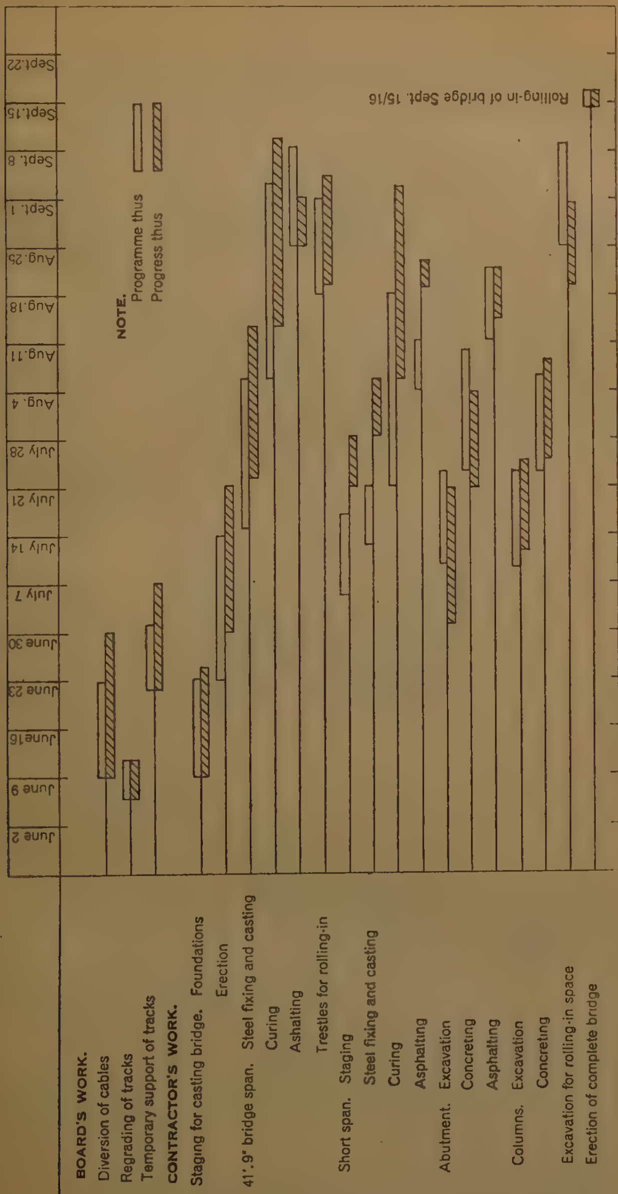
To provide for the possibility of lack of alignment, ½ inch of clearance between the lips on the column-tops had been allowed, but it was evident that, with the rolling-in arrangements provided, this was insufficient. A secondary rolling arrangement, giving a movement at right angles to the direction of rolling-in, would have been an improvement, although increasing the travel of the jacks. To dispense with the lipped column-top and to allow in design for a greater degree in eccentricity of loading would appear to be the simplest arrangement, although the lip has the advantage of guarding against the tendency of the bridge to creep over the tops of the columns.

After the 41-foot 9-inch span was correctly placed in position, bitumen was filled around the bearing on the abutment, and the permanent way was made good for the steam-crane to run over in order to place the 9-foot 6-inch spans at the west end. The joints at both ends of the 9-foot 6-inch spans were covered by copper strips, asphalted, and a protective coating of quick-setting concrete was laid over the joints. The permanent way was made good during the Sunday night, and the line was re-opened at the usual time on Monday morning.

Progress of Work.

The closing of the railway between South Harrow and Rayner's Lane stations on Sunday, 16 September, necessitated the operation of a special omnibus service between those two stations. On account of the special traffic arrangements it was particularly important carefully to plan the programme of work and to fix the date of the rolling-in well in advance. The programme and progress was plotted on a chart of a type which is a standard practice for new

Fig. 5.



Programme and Progress of Work.

works of the London Passenger Transport Board. An extract from the programme and progress-chart relating to the works in connexion with the bridge is shown in *Fig. 5.*

Quantities.

The 41-foot 9-inch span, complete with permanent-way slag, and one track, weighed 250 tons. The short slabs each weighed 11½ tons.

The amount of concrete, excluding the protective cover, was 2,826 cubic feet in the 41-foot 9-inch span, 327 cubic feet in the 9-foot 6-inch span, 81 cubic feet in the columns, and 729 cubic feet in the column bases. The amount of reinforcing steel was 424 cwt. in the 41-foot 9-inch span, 18·7 cwt. in the 9-foot 6-inch span and 25·4 cwt. in the columns.

An indication of the relative cost of the various classes of work included in the construction of the bridge-deck alone is given below. The percentages are based on the tender-prices, which, for this particular part of the work, closely followed the average rates submitted by most of the tenderers.

Work.	Class.	Proportion of total cost: per cent.
41-foot 9-inch span	Concrete	12·4
	Steel reinforcing	22·9
	Shuttering	12·8
	Asphalt and protective cover	6·1
	Erection, not including temporary support of tracks and necessary excavation . .	39·4
9-foot 6-inch span	Complete	6·4
	Total . .	100·0

Conclusion.

The work was carried out under the general supervision of Mr. A. R. Cooper, M. Inst. C.E., Chief Engineer to the London Passenger Transport Board, with whose permission this Paper is presented.

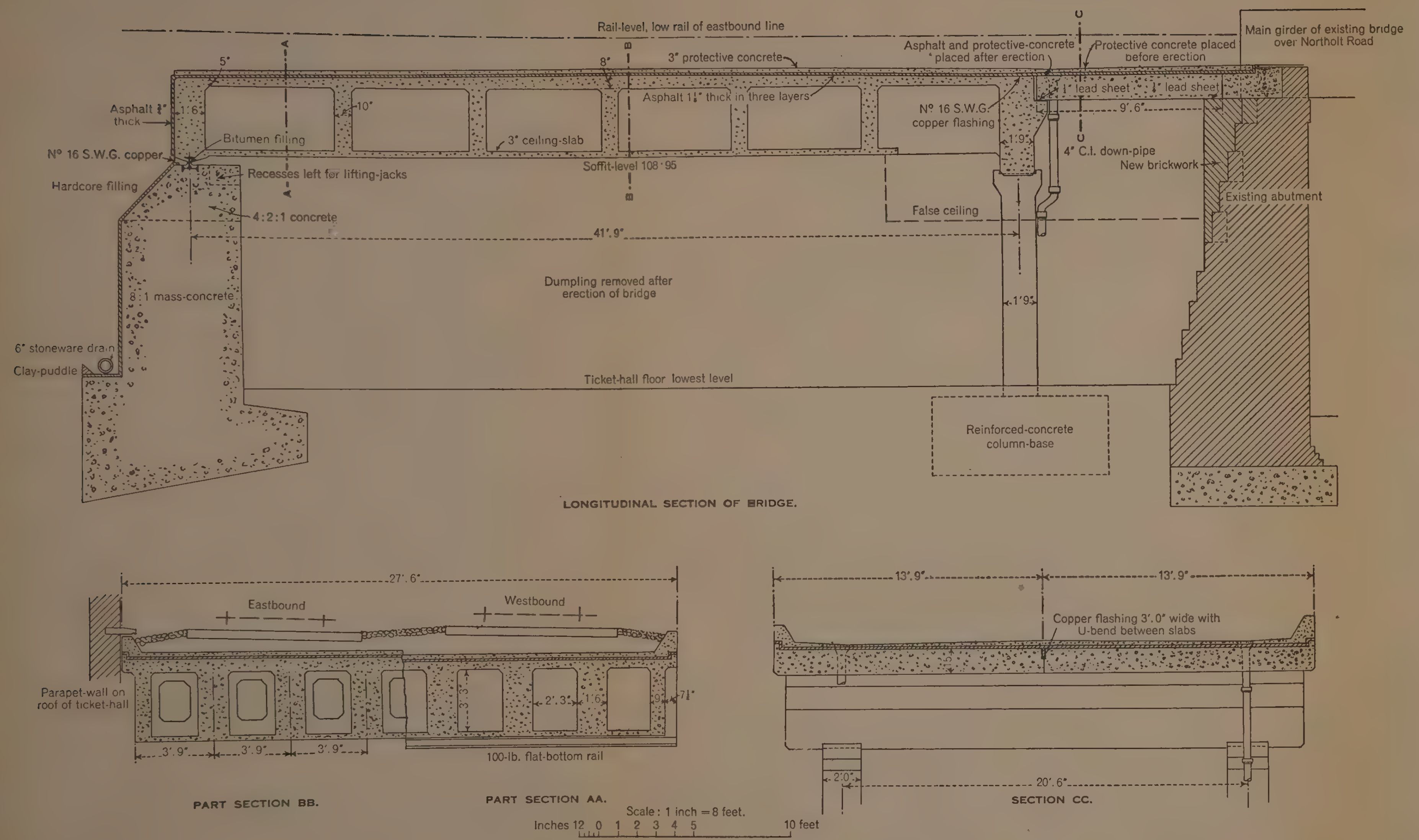
The Author assisted in the design of the bridge, prepared the contract drawings, and afterwards acted as Assistant Resident Engineer for the engineering work in connexion with the station reconstruction.

The general contractors for the work were Messrs. W. and C. French, and Messrs. Askham and Palin acted as sub-contractors for the rolling-in of the bridge.

The Paper is accompanied by four sheets of drawings and one diagram, from which Plate 1 and the Figures in the text have been prepared, and by four photographs.

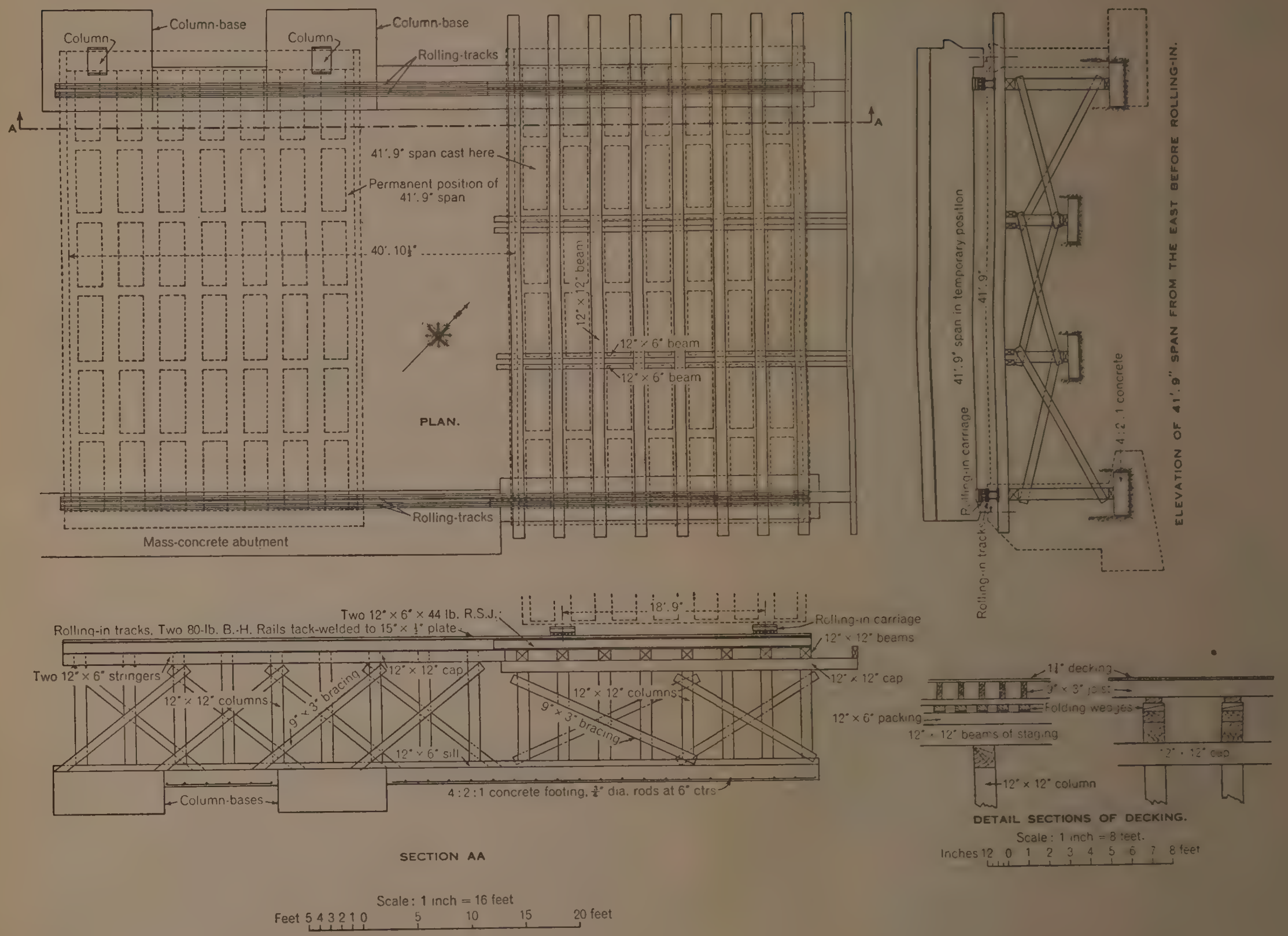
A PRE-CAST REINFORCED-CONCRETE UNDERLINE RAILWAY-BRIDGE.

Figs: 2.



NEW BRIDGE.

Figs: 3.



TEMPORARY STAGING AND TRESTLES FOR CASTING AND ROLLING-IN 41'. 9" SPAN.

H. G. FOLLENFANT.

Paper No. 5012.

"Improvements in the Harbour of Port Louis, Mauritius."

By Harold Cholmley Mansfield Austen, C.B.E., M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

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Introduction.

Port Louis is the capital and only commercial harbour of the Colony of Mauritius (*Fig. 1*, p. 36). The bay of Grand Port in the south-east of the island was the harbour used by the Dutch in the seventeenth century, the main settlement being Vieux Grand Port, but soon after the island had been annexed by the French in 1715, Mahé de la Bourdonnais, the virtual founder of the Colony, transferred the seat of government to Port Louis, at which place he built forts, quays, sheds, slipways, and even ships. From then until 1810, when the island was captured by the British, Mauritius, or Ile de France as it was called by the French, was a port of call for French ships trading to the east and a base for the French navy during their Indian campaigns.

Under British rule the trade of the Colony gradually developed into that concerned with the main product, which is sugar. Without any serious alterations except for the provision of quays for lighters, sugar-sheds, and fairly expensive dredging operations in 1907, the harbour remained much the same as 100 years ago until 1922, when

[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—Acting Sec. Inst. C.E.

a report envisaging an expenditure of £207,000 on extending the facilities for the landing and storage of goods by lighters and for dredging was drawn up by Messrs. Coode, Fitzmaurice, Wilson, and Mitchell in August of that year, following a period of serious con-

Fig. 1.

MAURITIUS.

gestion to shipping due to abnormally high sugar-prices and the general inflation of trading conditions. The Author, on the recommendation of this firm, was appointed Harbour Engineer by the Crown Agents, and reached the Colony in July, 1923.

The then Governor, Sir Hesketh Bell, G.C.M.G., who was on leave from May, 1922, to March, 1923, held the view, on his return to the

Colony, that very few additional facilities were required to accommodate the trade of the port under normal conditions, but that, having regard to what had recently occurred and to the flourishing state of the Colony's finances, the time had perhaps come to modernize the port in such a way as to enable cargoes to be handled at cheaper rates, and to provide up-to-date facilities for vessels wishing to land passengers or cargoes.

A good deal of local opposition to the proposal of the Government *inter alia* to build deep-water quays was put forward by the lighterage companies, who for many years had held the monopoly of harbour-traffic, and, although a scheme for building deep-water quays, reclaiming a shallow part of the harbour and dredging was adopted by the Council of Government in February, 1924, it was not until May, 1925, during the Governorship of Sir Herbert Read, K.C.M.G., C.B., that a modified scheme of works on these lines was actually commenced. The modified scheme included one deep-water quay, the dredging of the harbour to a depth of 32 feet at low water, and the reclamation of about 10 acres of valuable water-frontage with Government-railway access, all at a cost of £138,000 (Rs.2,070,000 at that time [1]). A plan of the harbour is shown in Fig. 2, Plate 1.

Dredging of Harbour.

The most urgent work to be done was the dredging of the harbour, no serious maintenance dredging having been done since 1907. It was considered advisable to avoid the repurchase of expensive plant and to utilize plant which the Colony could afford to run and maintain after the berths and channel had once been dredged to the depth ultimately required, namely 32 feet at L.W.S.T. An old 1-ton Priestman grab had some years earlier been fixed on a new wooden pontoon, the *Agrippa*; the latter was thoroughly overhauled and a modern 1-ton Priestman grab was fitted, capable of dredging to a depth of 40 feet. A second pontoon, the *Cæsar*, constructed mainly of teak, was built and was fitted with a 3-ton Priestman grab capable of dredging to a depth of 45 feet. One new 50-ton steel hopper-lighter was provided, two similar existing lighters were repaired, and a fourth was rebuilt. To facilitate the discharge of the lighters and to fill in the reclaimed area by pumping, a steel barge was bought and was fitted with two 24-HP. marine engines and propellers, a boiler, a condenser, and a 10-inch Gwynne pump. The barge actually used was one of two " A " barges that had been constructed during the War for the purpose of conveying munitions from Richborough to the Belgian coast. These barges

[1] Re.1 at that time was equivalent to 1*s*. 4*d*.

had been brought to the Colony in numbered pieces by Sir William Garthwaite, the owner of two local sugar-factories, but they had never been assembled. The vessel so constructed was called the *Pompey;* it was provided with a hinged suction-arm capable of pumping mud and sand from a depth of 32 feet, and with a wood-lined and caulked hold capable of containing some 50 tons of dredged material. Whilst a considerable area of harbour near the reclaimed area was dredged by means of the suction-arm, the mud-hold was seldom used; direct pumping from the harbour was also discontinued, partly on account of delays in adjusting the floating pipe-line for passing tugs and lighters, but mainly because the most economical use of the craft was proved to be the discharge of lighters by pumping their contents, including coral and coal up to 5 inches diameter, into the area reserved for reclamation. About 200,000 cubic yards of the dredged material were pumped out of the lighters by the *Pompey's* pump, but the greater part of the dredgings were taken out to sea and were dumped in deep water. The Port Department tugs and launches were used both for conveying the lighters to sea and alongside the *Pompey.*

Reclamation-Works.

The area reclaimed included a large patch of rock bottom covered with mud and sand with an average depth of about 6 feet of water over it. Some old wagons and a locomotive were hired from the Railway Department, stones and hard filling from the neighbouring Fanfaron fortifications being thereby conveyed to the site and tipped into the water to form a rubble embankment; the latter settled through the overlying soil to the rock-level about 9 or 10 feet below water-level, so as to form a border to the top of the slope of the rocky patch. As soon as the rubble embankment was closed, a masonry-faced parapet-wall on a reinforced-concrete slab foundation was built on the rubble embankment to retain the dredgings (*Fig. 3*). A 12-inch-diameter drain was formed across the centre of the reclaimed area with a tidal flap-outlet through the wall.

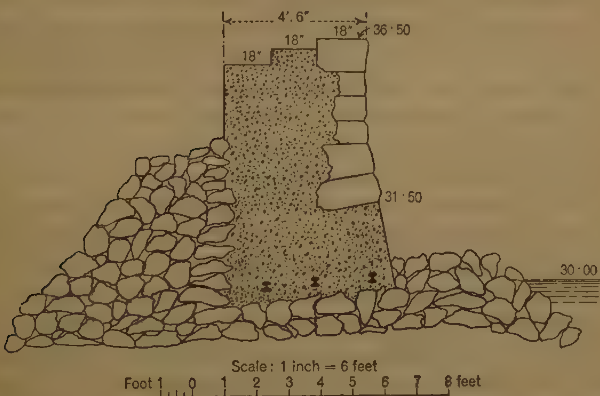
The wall on the north side of the reclaimed area, where the bottom was stiff sand overlying rock, was built on a piled foundation, the piles consisting of large numbers of old 21-foot 74-lb.-per-yard iron rails driven down to the rock. These rails were tied together by long lengths of old mooring-chains which had been picked up from the harbour-bottom. The concreting was done under the supervision of divers with the help of a crane, the concrete being laid in amongst the mass of rails and chains by means of a special hopper-bottomed skip with surface release. The broken stone used in the concrete, which was 6 : 1 with a liberal admixture of stone displacers, was

made to pass through a 1-inch mesh. The purpose of this wall was to provide a backing and anchorage attachment for the ferro-concrete piled deep-water quay, to be constructed later, parallel to its length.

In all cases where concrete was used in these works the sand was not the usual coral-sand found on the shore, but was specially-selected river-sand, every cubic yard of which was thoroughly washed with fresh water and hand-screened into heaps of gravel, grit and sand.

In carrying out the dredging some 500 tons of coal and patent fuel were taken up from the harbour-bottom, together with one hundred and twenty sleepers and a large variety of iron bars, scantlings, and other objects, which had interfered with the berths

Fig. 3.



and approach-channel. The recovered coal and patent fuel were sufficient to keep the dredgers and pumps at work for more than 2 years. Apart from mud and sand it was found necessary to break up and remove large quantities of coral. To do this another "A" barge was purchased, cut in half, and fitted with a powerful winch, boiler, cathead, chains, and trip-gear. An old 6-inch steel gun was obtained from the military authorities, its breech turned so as to form a rounded end and bored out to take a 9·2-inch armour-piercing shell, which was fitted into the bore by means of an adapter-rod passing through the muzzle and terminating in a heavy ring. The ring was shackled to the winch-chain and a keeper-chain fitted to the muzzle of the gun, which thus acted as a rock-breaker.

Deep-Water Quay.

Generally the quay was constructed as shown by the cross-section (Fig. 4, Plate 1). The original position of a coral-ledge extending

above the basaltic rock but about 20 feet below water-level is shown in Fig. 5, Plate 1. The coral, when borings were first taken, was found to be so impenetrable and so close to the basaltic rock which underlay the area to be reclaimed that it was at first thought to be solid enough to provide a foundation for the quay. The quay was therefore originally intended to be placed at an angle more nearly north-east and south-west. However, as soon as the mud and sand had been dredged to the level of the coral and the rock-breaker had been brought into play to "trim" the coral-frontage and to test its quality, it was found that the gun, which weighed 2½ tons, when dropped through a height of 20 feet either broke off or penetrated the coral-layer after from three to six blows had been delivered on the same spot; further it was found that sand lay underneath the coral. The rock-breaker was then systematically worked from south to north, each cut from the south tending in a more easterly direction until the borings under the site of the former coral-patch showed that the basaltic rock was at a suitable level for the pile-foundations of the quay. Actually, a very stubborn patch at the south end of the quay was left and an old steel lighter, measuring 45 feet by 15 feet and reinforced with concrete, was sunk upon it and filled with stones, 30-foot sheet-piles being driven around the seaward faces just clear of the coral-patch and tied back to the retaining wall in the manner shown in Fig. 5, Plate 1. The rock-slope seaward of these piles fell away somewhat rapidly, so that the 14-inch piles forming the south-west corner of the quay had to be made 55 feet long. These came to a set with their heads ranging from 1 to 5 feet above water-level. At the north-eastern end, as the reclamation-wall was being tipped, a dead end for the stone embankment was formed by sinking on a coral-patch (which had first been carefully levelled off by divers and was at a depth of 21 feet below water-level) two lighters joined together by steel girders in dry dock and reinforced with concrete. The positions of these lighters are shown in Fig. 5, Plate 1. Steel frames consisting of rolled steel joists were fitted all round the lighters so that, when sunk, their tops projected above water-level. Moulded concrete slabs were then let down between the joists, and the tipping of the stone embankment was proceeded with. The rock over the site off the north end of the quay and seaward of the sunken lighter was found at from 55 to 65 feet below water-level. As ferro-concrete piles of the required length could not be handled except at great expense, turpentine sheet-piles were driven around the patch forming the foundation of the lighters, and the intervening triangular space up to the line of the quay frontage, when dredged to 35 feet, was studded with turpentine piles 76 feet long, driven down to the rock. A reinforced-concrete slab 15 feet thick was laid round these

piles, which were well bedded into the surface of the slab, 20 feet below water-level. Ferro-concrete columns were built into this slab in continuation of the line of 14-inch piles forming the normal piled section of the wall. To safeguard against any possibility of outward lateral movement the whole north angle of the wall, both for the front and the return end, was, as in the case of the south-west end, tied back to land anchors by means of rods and chains enclosed in concrete.

The principal form of reinforcement used consisted of old 74-lb. double-headed steel rails, from 21 to 30 feet long, of which a very large quantity was available in the Railway Department at a price of R.1 per foot run. Neither the price nor the section were strictly economical, but it was desirable to reduce the stocks of old rails, and the practical results as reinforcement were entirely satisfactory. The decking was designed to carry a load of 1 ton per square foot, but otherwise there was nothing remarkable about the reinforced-concrete work. The aggregate consisted of the screened and washed river-sand and stone chippings with some river-gravel. British Standard Portland cement was used in the proportion of 1 part of cement to 4 of aggregate.

The 14-inch piles were of the ordinary square section, and, as already stated, many of those used in the front rows, being 55 feet long, weighed nearly 6 tons. The heaviest travelling crane available only had a lifting-capacity of 5 tons, and hence, after the pile had been conveyed to the site on three small bogies from the pile-yard, two cranes were required to place the piles within reach of the floating pile-engine. A girder was strapped to the pile along its centre to prevent the pile bending and the concrete cracking under the lift. The head of the pile was then raised by the pile-engine whilst the toe was slung by a crane, the latter's wire being paid out as the pile-head lifted until the pile was more or less plumb. The pile-engine and pile were then shifted to the site where the pile was to be pitched, and the pile driven. From four to six piles a day were thus driven, great care being exercised with the driving when it was evident from the borings taken that the toe must have nearly reached rock-level, in order to avoid distortion of the pile. Towards the centre of the quay the face-piles were specially packed around with concrete in bags.

The object of the deep beam-and-masonry wall surmounted by a stone coping to form the face of the quay was to prevent damage on the rare occasions when a strong northerly wind or a cyclone would cause a run of sea against the quay.

The fenders used were originally of the circular rope-wound wooden variety suspended vertically around a sheave from springs

fitted within the wall. These were subsequently replaced by similar (although larger) horizontal fenders. The latter have finally, on grounds of economy, been replaced by horizontal fenders, 8 feet long by 3 feet in diameter, made of thin branches of a local wood called *goyave de chine*, which is similar to hazelwood, the branches being individually wired together and the whole bound round a central core of stout wire formed into eyes at each end, the eyes being shackled to chains suspended from the sheaves in the quay face.

A tide-gauge was fitted in the quay-office, the float of the gauge being in a specially-protected chamber formed of a cast-iron pipe under the south-east corner of the quay. Graphs recording the behaviour of the tides under cyclonic storms have been kept.

The quay was intended for the landing of petroleum-products in barrels and cases, coal, sleepers, and such other cargoes as could conveniently be discharged at this part of the harbour. Road and rail access is provided, and the whole of the works were completed during 4 years within the figure estimated. A wagon-weighbridge for weighing coal and other materials was installed on the approach-siding clear of the quay. The first ship containing 1,156 tons of petroleum-products was discharged at the quay from the 15th to the 19th April, 1929.

Working of Quay.

As there was no Government commercial authority with experience of port management, the duties of working the quay's traffic fell to the Author. Although the quay has never been used to its maximum capacity in deference to the opposition of the lighterage companies and the consequent limitation of Government activity in the harbour, the results which have been obtained after 8 years' full working are shown in Table I.

There are no cranes on the quay, and the ships' own gear is used in all cases. The rate of discharging coal by lighter from ships in the port used to be about 400 tons a day, whilst the average rate of discharging at the quay is 670 tons a day, with a maximum rate of 1,100 tons per day. The rate of discharging and stacking petrol cases into store from ships by lighters used to be about 8,000 cases a day (20 cases weighing 1 ton), but at the quay the average daily rate is 13,500 cases, with a maximum rate of 19,600 cases per day. The percentage of cases requiring ullage has fallen from about 3 to under ½ since the quay was commissioned.

TABLE I.—HARBOUR TRAFFIC RESULTS FOR THE YEARS 1929–1937.

QUAYS "C" AND "D": RECLAMATION AND PART-DREDGING COST Rs.1,200,000.

Year.	Ships.	Tons handled.		Savings on pre-quay costs to consignees (both Government and private): Rs.	Gross revenue: Rs.	Expenditure: Rs.	Net profit on working combined with saving to Government departments: Rs.	Return on capital outlay: per cent.
1929–30	17	Petroleum products	6,520					
		Coal	35,310					
			41,830	54,548	129,974	89,944	76,165	6·3
1930–31	18	Petroleum products	9,772					
		Sleepers	782					
		Coal	31,307					
		General	2,716					
			44,577	71,760	131,426	81,048	90,433	7·5
1931–32	22	Petroleum products	10,357					
		Coal	19,345					
		General	2,722					
		Cattle	127					
			32,451	48,499	95,590	57,889	64,445	5·3
1932–33	34	Petroleum products	12,974					
		Coal	12,942					
		Sleepers	1,362					
		Guano	1,480					
		Bitumen	222					
		Cattle	210					
		Fish	62					
			29,252	46,863	87,629	50,727	60,984	4·7
1933–34	36	Petroleum products	12,191	With the above expenditure increased to Rs.1,375,000 by the inclusion of the partly-finished quay "C" and additional dredging.				
		Coal	28,744					
		Sleepers	1,672					
		Guano	1,847					
		General	2,182					
		Fish	141					
		Cattle	7					
			46,784	77,274	131,348	63,006	123,198	8·2
1934–35	39	Petroleum products	12,346	With the above expenditure increased to Rs.1,500,000 by the inclusion of quay "C," a larger proportion of the reclaimed area and additional dredging.				
		Coal	13,860					
		Sleepers	1,641					
		Guano	1,719					
		Cement	600					
		Bitumen	441					
		Fish	268					
		Sand	107					
		General	52					
		Cattle	52					
			31,086	49,609	94,559	54,083	68,522	4·5
1935–36	39	Petroleum products	12,463					
		Coal	20,259					
		Sleepers	813					
		Cement	720					
		Asphalt	589					
		Guano	2,845					
		Fish	219					
		General	72					
		Cattle	60					
			38,040	62,733	109,625	60,236	88,103	5·9
1936–37	37	Petroleum products	14,630					
		Coal	21,817					
		Sleepers	1,473					
		Asphalt	1,175					
		Guano	1,468					
		Fish	209					
		General	20					
			40,792	67,876	123,294	67,393	96,719	6·4

Additional Quay.

Another ferro-concrete deep-water quay known as Quay "C," which provides 17 feet of water (and is capable of providing 25 feet by dredging), was completed in 1931. Its cost was Rs.225,000 and its position is alongside the old wharf separating the Trou Fanfaron from the outer harbour. Its principal use is for landing light cargoes, cattle, etc. A small but regular trade is done at this quay, consisting at present of guano, salt fish, etc., usually brought in sailing vessels from the island dependencies of Mauritius.

Granary.

The late Sir Andrew (then Dr.) Balfour, the eminent health expert, who visited the Colony in 1921, strongly advocated the elimination of the numerous grain-stores in Port Louis, as one of the most effective ways of combating the spread of plague which had often threatened the inhabitants, many of whom were then being carried off yearly by the disease.

Expert opinion held that plague was disseminated by the presence of rats and their fleas in the grain go-downs around the seaward areas of Port Louis. It was not, however, until April, 1924, that the question of erecting a rat-proof granary was definitely considered. A committee was then appointed by the Governor, Sir Herbert Read, K.C.M.G., C.B., which decided that the building should be capable of storing roughly 4 months' food-supply or 300,000 bags of rice, dholl, ghee, lentils, etc. A year later, a preliminary expenditure of Rs.100,000 was approved by the Secretary of State, and in February, 1925, the Author's proposal to erect the granary on the site of the Port Department buildings, in the position indicated in Fig. 2, Plate 1, was approved at a cost of Rs.2,460,000, in spite of strong local opposition on the score that such a building would be a threat to vested interests, that the nature of the project was experimental, and that the economic difficulties of the island did not justify the expenditure. Final authority to proceed with the work was given in August, 1926. By March, 1927, the old Port Department slips and buildings had been removed and new accommodation on improved lines had been supplied elsewhere. The building of the granary itself was thereupon commenced.

The granary is wedge-shaped, being 390 feet long on the seaward side, 209 feet long at the south end, and 108 feet long at the north end, and is 72 feet high. Bags of rice can be stored thirty-three high on the ground floor and twenty-five high on the first floor. The top

floor is used, as a general rule, exclusively for sorting, the merchants' stores all being on the first and ground floors.

Practically the whole area to be occupied by the granary was of coral formation. This was found to be 10 feet deep at the back, where the road and railway siding run, and at the south end next the Port and Customs office, but to taper off to a few feet near the sea-frontage and towards the north end. The upper layer of coral, on which formerly all the Port stores and shops had been built, was very hard, but just above the level where the red earth, overlying boulder clay, was reached, the coral was of a more floury and soft nature. At no point on this coral-bank, when excavated to floor-level, would it have been safe to lay foundations for a building of the size and weight of the granary. A large number of pits had therefore to be pierced through the coral, in order to excavate the subsoil down to the hard boulder-clay where a solid foundation was procurable. To make these pits and to remove the coral-bank at the west front and south end of the granary, an Ingersoll-Rand rock-drill was at first used, but experience soon proved that coral was responsive neither to mechanical drilling nor to blasting powder, and better results at cheaper rates were obtained by the use of manual labour (a labourer's wage was then R.1·25 per day), with hammers and steel wedges. The pits were filled with 6 : 1 mass-concrete 8 feet square at the bottom and stepped up to 5 feet square at the level of the feet of the columns. Owing to the difficulty of reaching the boulder clay by sinking pits along the line of the front or seaward row of columns, and in the case of most of the second row of columns, due to the expense of poling and pumping, ferro-concrete piles were driven down to rock-level. The piles were surmounted by ferro-concrete slabs tied together longitudinally and laterally.

The main feature of the granary is perhaps the ferro-concrete plinth which entirely surrounds the building 4 feet above the road level, and projects 9 inches from the face of the wall. It is held that a rat cannot jump 4 feet from the ground or run up a wall this height and climb over the projection. No water is laid on and no liquid or food is allowed to be taken into the granary. The yard surrounding the granary is paved with flat stone slabs, fine-punched and laid to a true gradient, and properly drained, so that no water can lie near the building. Access to the building at different points is given by means of short iron ladders, hinged above the plinth-course, so as to cut off ground-communication. The first and top floors of ferro-concrete are reached by means of three man-elevators, consisting of small hinged platforms and hand grips attached to a continuous vertical belt extending from the ground floor to the top of the building through holes in the various floors. Rapid exit is

secured independently of the man-elevators by means of three sets of brass sliding-poles. The spaces between the ferro-concrete columns forming the outer walls of the building were filled with a double row of brickwork in cement, the bricks all being obtained from the local factory, and the core being stiffened with hoop-iron taken from cement-barrels.

Working of Granary.

In discharging the grain the bags are conveyed out of lighters, which lie alongside the small ferro-concrete quay in front of the granary, by means of six hinged adjustable conveyor-arms, without touching the quay. These six conveyors shoot the bags through the wall of the granary on to six vertical elevators, by which the bags are carried to the top floor, where they are discharged on to a platform and man-carried to stacks, usually eight bags high, and sorted under their various and numerous marks. Customs clearance is given, after duty has been paid, at this point. The bags are then carried by small rubber-tired 1-ton hand trucks to the manholes, fixed flush with the floor, which give access to the various stores. Thence the bags are carried by spiral shoots down into the merchants' stores, and stacked under their appropriate marks. The bags are removed to railway wagons, lorries, or carts from the top and first floors by means of seven "Swiftsure" shoots and from the ground floor through the vertical rolling doors at the level of the plinth. The maximum load carried by the first floor is about 5 cwt. per square foot and by the top floor about 3 cwt. per square foot. The building is lit throughout with electric light and all the machinery is operated electrically. The machinery and electrical equipment were supplied by Messrs. Spencer (Melksham), Ltd.

The granary was completed in February, 1931, but was not put into commission, for political reasons, until April, 1933; 18,000 bags a day have been conveyed into and stacked in the granary, and the system of working, in spite of local opinion, has proved entirely satisfactory. As compared to storing, as formerly, in numerous go-downs in the town, grain-ships now secure quicker dispatch, the handling costs are less, the condition of the grain, stored in a clean building, is superior, and pilferage is practically eliminated. The work was completed within the estimated cost.

The handling-charges and rent of stores within the granary are levied from the merchants by means of a tonnage rate irrespective of the time during which the grain is stored. This works out quite satisfactorily and is much simpler than the pre-granary method of adding to the cost of the grain 2 or 3 cents per bag per month for

storage. The charges are so fixed as to produce a profit equal to slightly over 3 per cent. on the capital outlay.

Customs Warehouse Widening.

The Customs warehouse extension was carried out at a cost of Rs.60,000 in order to avoid the carting of surplus goods to a more remote store. This work effected a saving of about Rs.20,000 annually.

Conclusion.

The costs of the various works are shown in the Appendix.

All the works were designed and carried out with local labour by the Author, who has also been responsible for the working of both quays and granary on commercial lines. The entire staff was Mauritian, amongst whom Messrs. G. Blackburn, R. Nicolin, and W. Lebret deserve special mention.

The Paper is accompanied by five sheets of drawings, from some of which Plate 1 and the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Statement showing Estimated Cost of Works.

(1) The works undertaken under the original scheme costing £138,000 (Rs.2,070,000) were :—

	Rs.	Rs.
Quay "D"	675,000	
Dredging, embankment and siding	1,005,000	
Silt-traps, maintenance plant	180,000	
Salaries and various other charges	210,000	
		2,070,000

(2) Extras to original scheme were :—

Coal-equipment	210,000	
Additional dredging	120,000	
Four stores	30,000	
Weighbridge	30,000	
		390,000

(3) Works subsequently undertaken were :—

	Rs.
Quay "C"	225,000
Granary	2,460,000
Customs warehouse widening	60,000
Grand Total	5,205,000

(4) Details of works allocated to Quay "D" (*see* Table I) :—

	Rs.
(*a*) Quay "D" construction cost	675,000
(*b*) Part dredging, allocation from original dredging scheme	100,000
(*c*) Additional dredging	120,000
(*d*) Coal-equipment[1]	210,000
(*e*) Original sidings, new lay-outs, shed-accommodation, etc.	65,000
(*f*) Weighbridge	30,000
	1,200,000

(5) Dredging, embankment, and sidings (Rs.1,005,000).
This figure is subdivided as under :—

	Rs.
Dredging	870,000
Embankment walls	95,000
Original siding lay-outs	40,000
Total	1,005,000

[1] Not used owing to private merchants' coal not being discharged at the quay, and Government coal all being transferred to railway yards direct by rail from the quay.

(6) Detail of dredging expenditure (Rs.870,000, as above).
This figure is subdivided as under :—

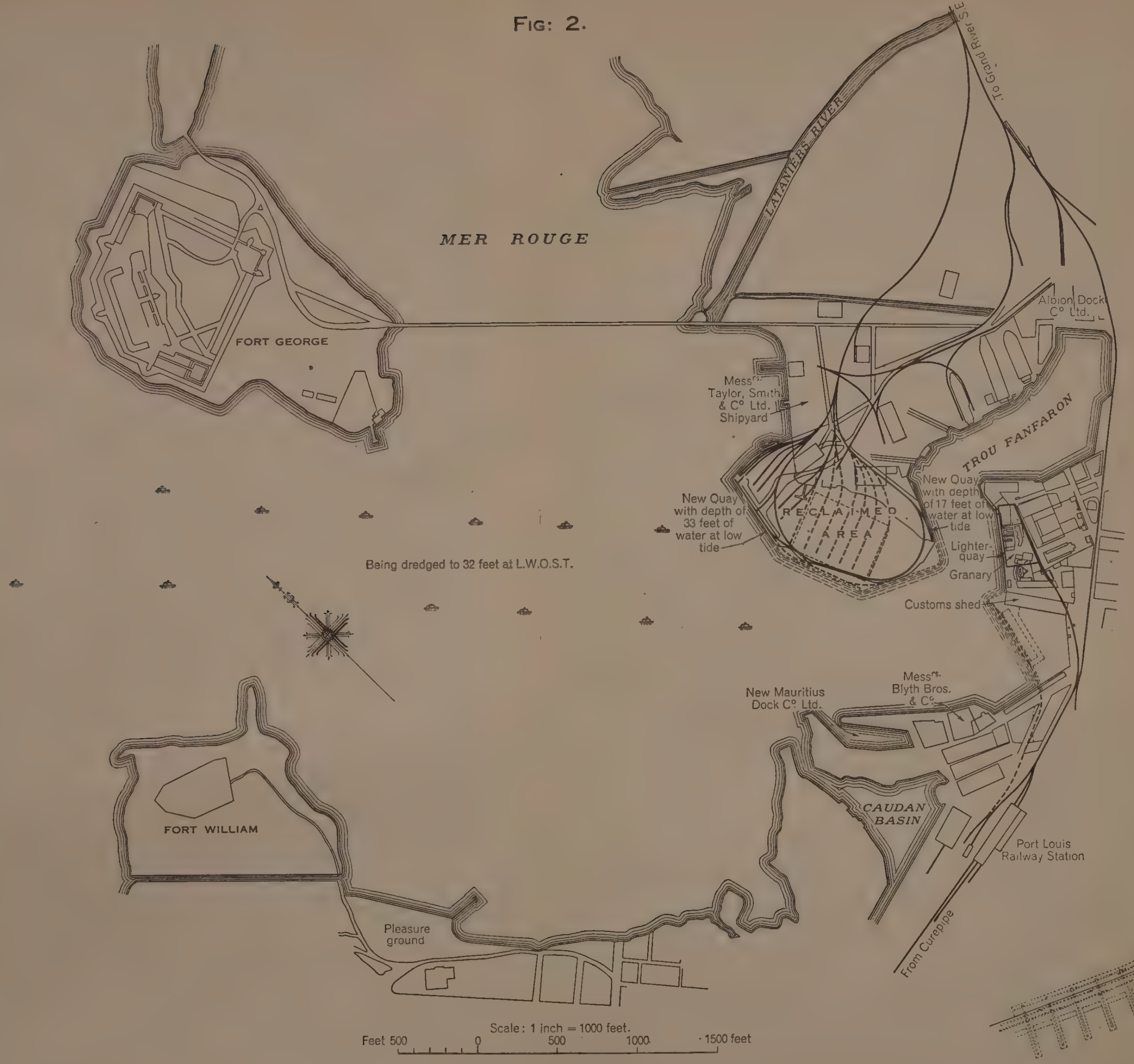
	Rs.
(*a*) Reconditioning of existing plant, purchase of new plant, erection-costs, spares for plant, and cost of annual overhauls and repairs	499,192
(*b*) Salaries, wages, and working expenses	370,000
Total	870,000

(7) Costs of the various plant-items used were :—

	Rs.
(*a*) *Pompey*	149,300
(*b*) *Cæsar*	125,317
(*c*) *Agrippa* (repairs to barge, new grab and spares, etc.)	43,156
(*d*) One hopper-lighter rebuilt	11,300
(*e*) One new hopper-lighter	28,650
(*f*) One steam mooring lighter and rock-breaker	56,850
(*g*) One pile-driver	40,290
(*h*) Two locomotive cranes	21,550
(*i*) One locomotive	11,625
Total	488,038

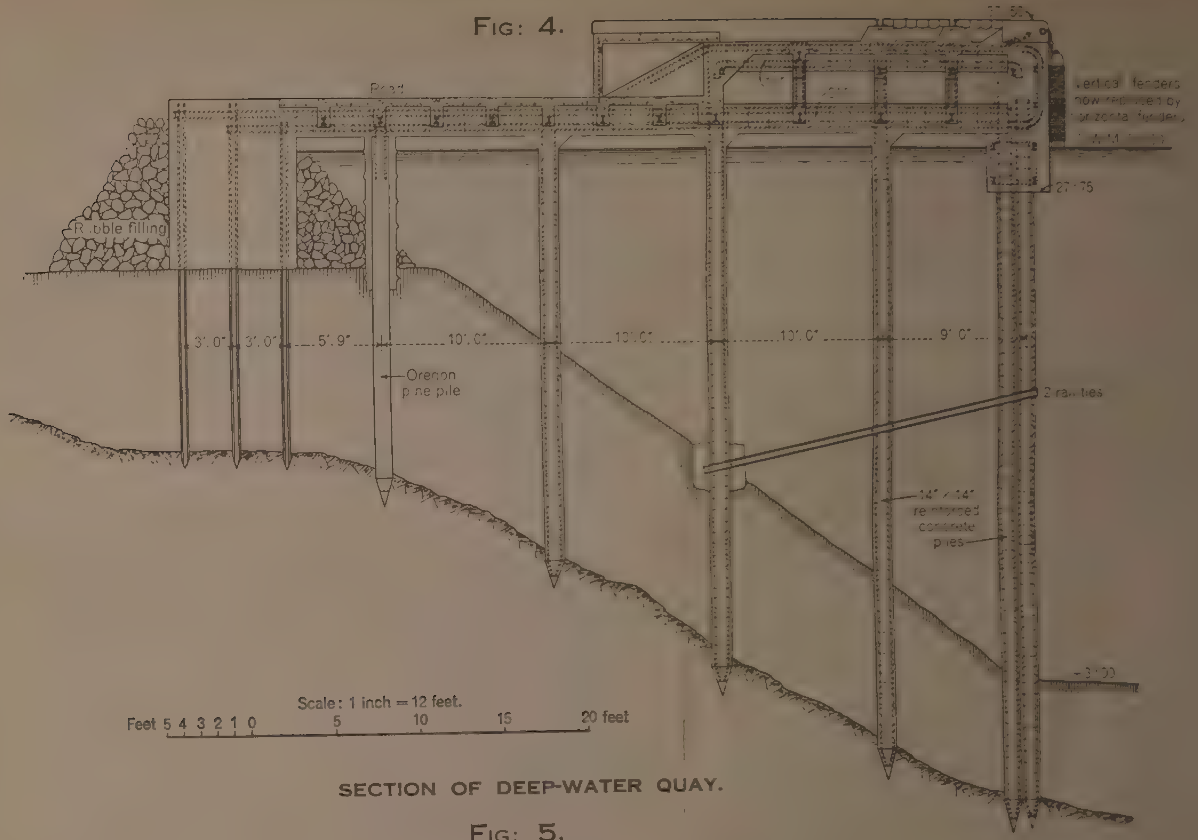
IMPROVEMENTS IN THE HARBOUR OF PORT LOUIS, MAURITIUS.

FIG: 2.



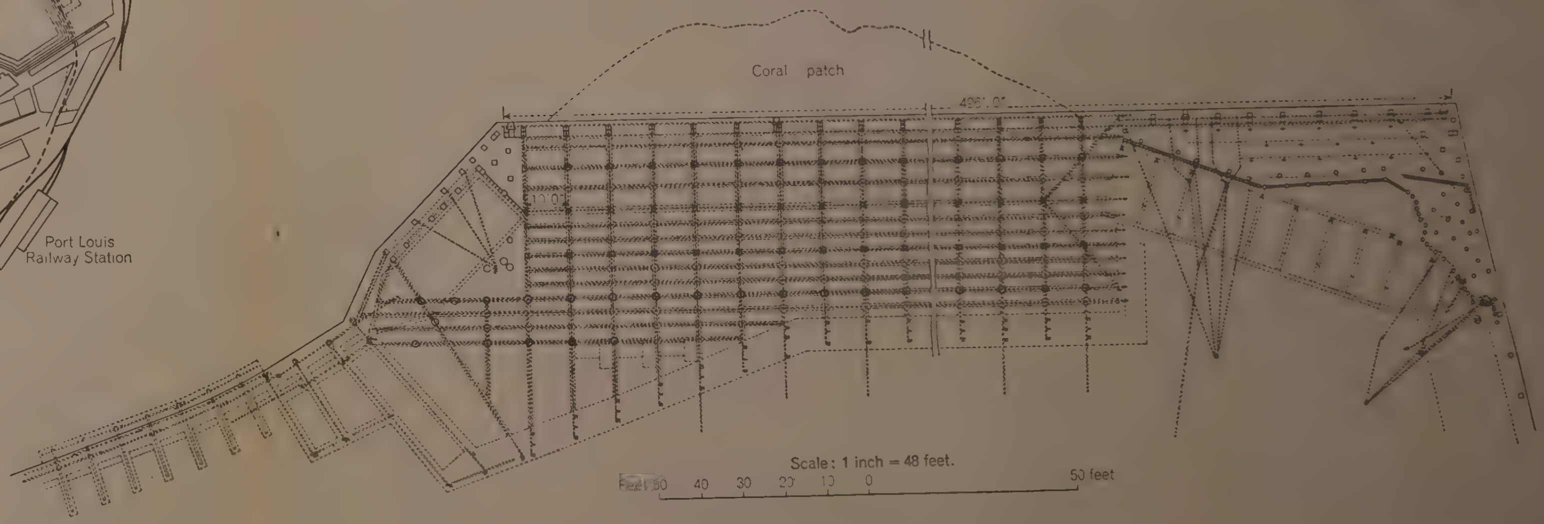
PLAN OF HARBOUR.

FIG: 4.



SECTION OF DEEP-WATER QUAY.

FIG: 5.



PLAN OF DEEP-WATER QUAY.

H. C. M. AUSTEN.

Paper No. 5094.

"The Reconstruction of the Inchcape Bridge, Bengal and North Western Railway."

By Douglas Walter Ravenhill and Guthlac Wilson, B.Sc., Assoc. MM. Inst. C.E.

(Ordered by the Council to be published with written discussion.) [1]

TABLE OF CONTENTS.

Introduction.

The Gogra river is one of the largest tributaries of the Ganges, and rises beyond the Himalayas in Tibet. About 500 miles from its source and 20 miles above its confluence with the Ganges it is spanned by the Inchcape bridge.

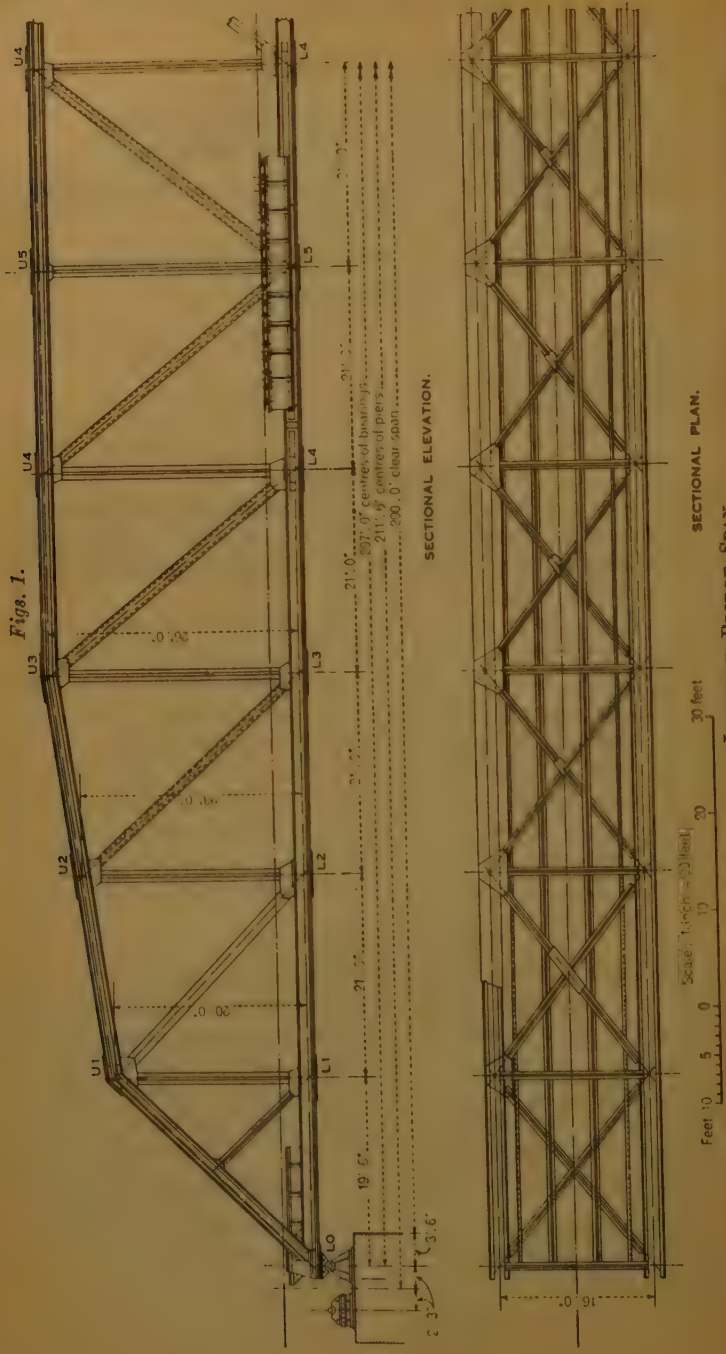
This bridge, built in 1909–10, consists of eighteen 200-foot through girder spans on brick piers, 36 feet high above low water-level (43 feet to rail-level) founded on circular brick wells, 26 feet 4 inches in diameter, sunk 97 feet below low-water level. The bridge carries a single metre-gauge track, and each span (*Figs. 1*, p. 50), complete with floor and permanent way, weighs 220 tons. The spans rest on knuckle-pins carried on cast-iron bearings of the usual type, fixed at one end of the span and resting on rollers at the other. The knuckle-pins are $8\frac{1}{4}$ inches in diameter, and have rims $\frac{1}{4}$ inch high at their ends to prevent lateral movement.

Earthquake Damage.

In the Bihar earthquake of the 15th January, 1934, two of the girders were overthrown and fell into the dry bed of the river. It is clear that the piers oscillated violently in a direction at right angles to the bridge. The period of the earthquake-vibrations was about $1\frac{1}{2}$ second, according to official records, and the acceleration about 5 feet per second per second. The inertia of the girders was so great that they could not follow this rapid oscillation, and they were only prevented from sliding to and fro along the knuckle-pins by the rims

[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—Acting Sec. Inst. C.E.

Figs. 1.



INCHCAPE BRIDGE SPAN.

at the ends of the pins. In some cases the rims were damaged and the saddles appear to have mounted them. In other cases the ends of the cast-iron saddles fractured on or against the rims. A typical case is illustrated by *Fig. 2* (facing p. 52), which is a photograph of the upstream bearing of span 17 on pier 16.

On two piers, where neither the rims nor the saddles gave way, the lower bearings, together with the masonry supporting them, were torn away from the body of the pier. In one case (pier 16) the masonry did not fall but cracked widely (*Fig. 3*, facing p. 52), but, in the case of pier 15, a slice of the pier 34 feet in height fell to the ground, carrying two bearings with it, which allowed the ends of the two girders supported by that pier to roll sideways and topple over into the dry river-bed (*Fig. 4*, facing p. 53). The other ends of these two spans hung on their piers and did not fall. The ends which struck the ground were badly crumpled, but the other halves of the girders were little damaged.

Figs. 5 (facing p. 53) show typical results of overstrain in the damaged parts of the spans at joints, as well as the flaking-off of paint on diagonal lines connecting rivet-holes. It is not clear why the paint should have flaked off only along these lines. The tension would not be expected to be much greater there than elsewhere, but obviously the stress was very different from what it was in the rest of the plate. Some shear-stress would occur along these diagonal lines and would be absent elsewhere, and that might be the cause of the curious result. The point may be of interest to designers of riveted joints.

The engineers who first saw the damaged bearings were led to believe from the nature of the fractures that the spans had actually rocked on their bearings, and that the cast-iron saddles had fractured through hammering on the rims of the knuckle-pins; this idea has not yet been entirely dispelled.

The Authors have attempted to investigate the problem mathematically and to determine whether the amplitude of the earthquake-vibrations could have been increased by the oscillation of the pier and span sufficiently to account for the spans rocking on their bearings, but they find that if the official figure for the period of the earthquake-vibrations is taken ($1\frac{1}{2}$ second, as reported by the Geological Survey Department of India) it is impossible to arrive at a horizontal acceleration great enough to cause rocking.

One of the Authors, however, who was in the district during the earthquake, noticed very distinctly that the vibrations at the surface of the ground had a period of $\frac{1}{4}$ to $\frac{1}{3}$ second, and made a note of this fact at the time. Furthermore, the Preliminary Report of the Geological Survey of India mentioned that many observers described

surface waves as undulations with a wave-length of 6 to 12 feet, which were apparently mere local effects of the earthquake and distinct from the "long waves" recorded by seismographs. If such short waves coincided with the natural period of oscillation of the piers and spans it is possible that the spans did actually rock on their bearings.

Conditions Governing Reconstruction.

The reconstruction of this bridge was a matter of great urgency, as it is on the Chupra—Benares main line which carries a heavy volume of traffic. Pending reconstruction a ferry was arranged for the transhipment of passengers, but goods traffic had to go round via Bhatni, an extra lead of 70 miles.

It would have been impossible to have new steel spans fabricated in time for the reconstruction of the bridge to be completed before the rains in June. The railway authorities were fortunate in having one exactly similar span in a disused bridge at Bagaha, 205 miles away, and they decided to try to make up the other span from the undamaged members of the two fallen spans. Serious doubts were expressed by the Senior Government Inspector of Railways, and also by others, as to the feasibility and safety of this scheme, but after completion and testing of the composite span he handsomely acknowledged it to be a fine and successful piece of work.

Quotations for the reconstruction were invited from firms of standing, and on the 9th February, 1934, the contract was placed with Messrs. Braithwaite & Co. (India), Ltd., who undertook to complete the work before the rains.

The work to be done involved dismantling the fallen spans 15 and 16; dismantling the broken parts in piers 14, 15 and 16 and supporting the ends of spans 14 and 17 while this was done; dismantling the span at Bagaha and re-erecting it at the Inchcape site; and assembling a second span from undamaged members of spans 15 and 16 and about 15 tons of new material which had to be fabricated in Calcutta. The rebuilding of the piers was done by the Railway under separate contract. A number of the bearings of other spans were broken and had to be replaced.

It was fortunate that the fallen spans were on the dry side of the river-bed, but it was to be expected that the river would rise and flood its whole bed by about the 20th or 25th of June.

Reconstruction.

The Contractor's plan for the work was to use unit steel trestles which could at first be arranged to support the spans to be dismantled and later rearranged as staging on which to re-erect the spans. Two

Fig. 2.

BEARING OF SPAN 17 ON PIER 16.

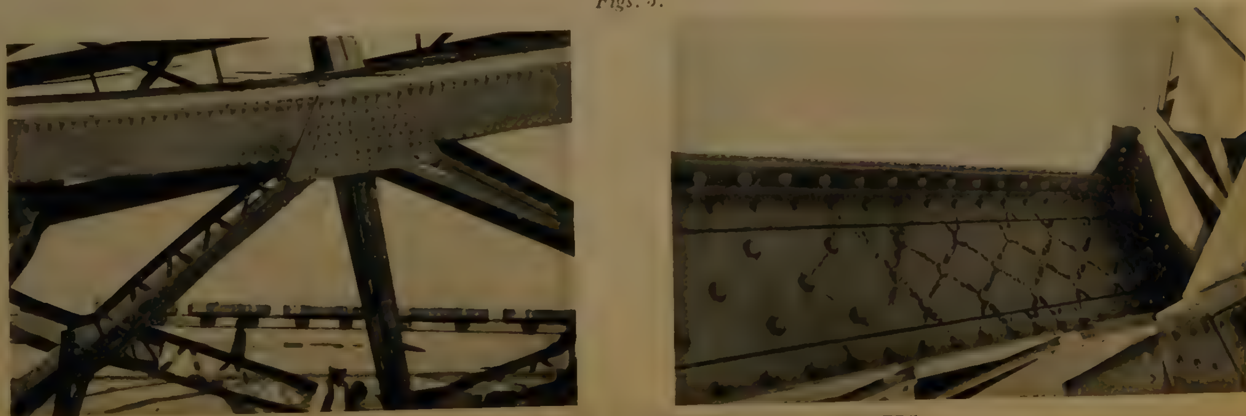
Fig. 3.

PIER 16.

Fig. 4.

COLLAPSE OF TWO SPANS DUE TO FRACTURE OF PIER.

Figs. 5.



FLAKING OF PAINT DUE TO OVERSTRAIN OF MEMBERS.

scotch derrick-cranes, running on tracks alongside the bridge, one upstream and one downstream, would handle the materials. The Railway provided an access-siding to the river-bed alongside one of the crane-tracks.

A gang of riveters with a supervisor left Calcutta on the 13th and started work at the site on the 15th February, propping up span 16 on sleeper cribs, as it appeared to be most unsafe, and cutting out the rivets of the trough decking in span 15, which was so firmly wedged against pier 14 as to be quite safe.

Ten days later, when all arrangements for dispatch of plant had been made, the Contractor's Engineer left Calcutta for the site. The unit steel "cube" staging was already being fabricated in the shops: this consisted of 5-foot cubes having vertical sides of 6-inch by 6-inch by $\frac{1}{2}$-inch angles, horizontal sides of 4-inch by 4-inch by $\frac{3}{8}$-inch angles, and flat-bar diagonal cross bracing on the sides. A certain amount of existing trestle staging, which was available in the Contractor's yard, was also worked into the scheme.

Unfortunately there were serious delays in getting the plant to the site, and plant did not begin to arrive at the site until the 23rd March, whilst work did not get properly started until the second week in April.

Meanwhile the Bagaha span had been supported on sleeper cribs and was being dismantled, and a careful survey was made of the two fallen spans at Inchcape to determine what material from them could be used. It was found that about 15 tons of new material would be required, comprising lower chords L_0L_2 and L_8L_{10} of the upstream girder, upper chords U_3U_5 of both girders, three panels of bottom laterals, and one panel of top laterals. *Figs. 6* (p. 54) show the make-up of the new span.

All plant had arrived by the middle of April, and thenceforward work proceeded steadily. By that time span 17 had been supported and the dismantling of pier 16 had been commenced, the damaged portion of pier 15 having been completely dismantled. By the middle of May both fallen spans had been completely dismantled, the span that had been dismantled at Bagaha had arrived, and trestle staging for its erection in the place of span 15 was nearly ready.

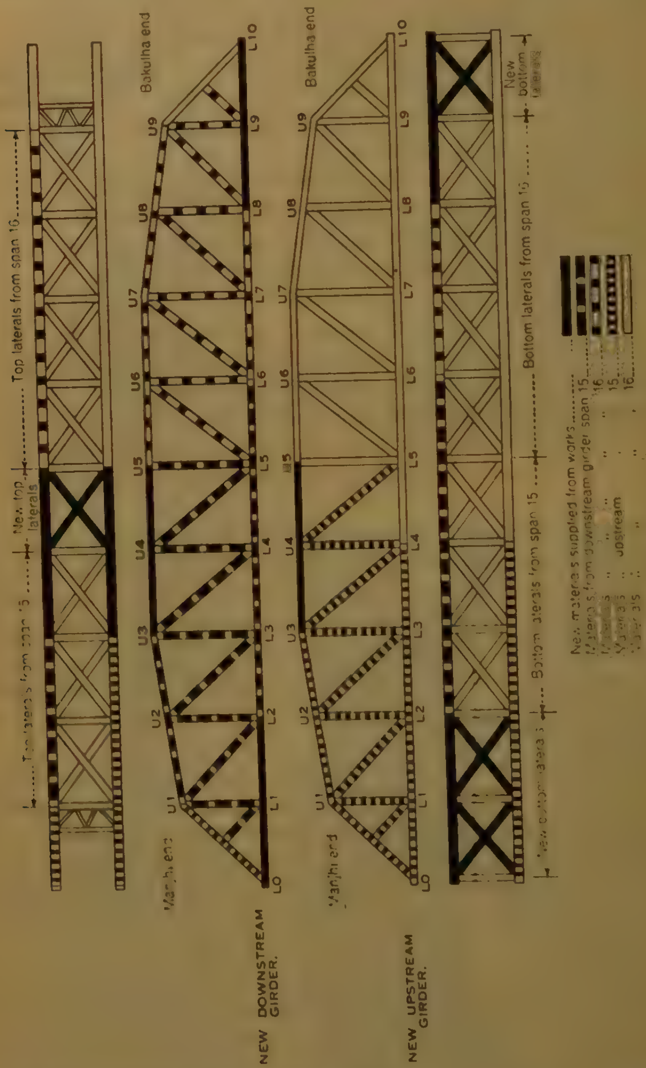
Severe sandstorms had been of almost daily occurrence since the beginning of April, and had hampered the work considerably, as they usually sprang up at about 10 a.m. and blew on into the afternoon. Arrangements were therefore made to illuminate the site with petrol lamps and to work two shifts, one from 4 p.m. to 1 a.m. and one from 1 a.m. to 10 a.m., starting from the 14th May.

By the end of May span 15 had been completely erected with the exception of the trough flooring and top lateral bracing, and riveting

was also complete except for the top boom joints. Staging for the erection of span 16 was well in hand.

Water-level in the river was now about 10 feet below ground-level

Figs. 6.



MAKE-UP OF REBUILT SPAN.

in the working-area, and it was expected that the river would rise this amount by about the 20th or 25th of June. Arrangements were made for the readings of the river-gauge at Chowka Ghat, 277 miles upstream of the bridge, to be transmitted by telegram daily. A

flood takes from 48 to 96 hours to reach the bridge-site from Chowka Ghat.

By the 10th June the floor-system and main girders of span 16 had been erected. A few rivet-holes in members of the fallen spans which were built into this span were found to have become elongated owing to the shock of the fall, and these were filled up solid by acetylene welding and re-drilled out to the correct size. After the fallen spans had been dismantled, two vertical members which it had been decided to utilize were found to be slightly bent; they were taken to pieces, the individual parts straightened, and the members rebuilt. Every joint of the trusses of this span was first assembled on the ground and checked over carefully by a skilled shop-foreman from the Contractors' Calcutta works. The new boom pieces were sent from Calcutta with the joint-holes drilled ½ inch in diameter; most of these were opened out to full size when the joint was assembled on the ground, and the rest after erection on staging.

By the 23rd June span 15 had been completed and all staging had been removed, and riveting of span 16 was well in hand. The river had now risen to within 6 feet of the level of the working-area.

By the 8th July both spans were complete and the bridge was ready for test. The river had risen so far that the working-area was now awash, and it had been necessary to concentrate on getting the plant and tackle out of the river-bed during the previous week: a week later the water-level was 6 feet over the working-area.

Tests of Rebuilt Spans.

On completion of the work the two newly-erected spans were thoroughly tested, and tests were also made on a third undamaged span for the purpose of comparison. The results showed no difference between the three spans.

The test train consisted of two "YB" engines and tenders and four loaded 12-ton trucks. Their total weight was 236 tons on a length of 207 feet. A brake-van was attached for the convenience of the guard, but this was clear of the span during the standing tests. No train was allowed on to the spans before the tests were made, so that the permanent set could be ascertained. When the test train was run slowly on to the reconstructed span and brought to a stand the deflexion at the middle of the span was 1·1 inch, and when the train was run off again a permanent set of 0·06 inch remained. Similarly, the test train standing at the middle of the span brought from Bagaha caused a deflexion of 1·2 inch, and on removal of the train a permanent set of 0·1 inch remained.

Similar tests on an old undamaged span showed a deflexion of 1·05 inch only. There was, naturally, no permanent set in this case.

The principal members of both girders of both spans were then tested with two Fereday-Palmer extensometers, and one girder of an undamaged span was also tested for comparison. The tests were made with the train standing, also at speeds of 5 and 20 miles per hour and at full speed, which varied from 45 to 60 miles per hour. The speeds were timed with a stop-watch. The results for the different spans were in good agreement with one another, and there was no indication that the span made up from the two fallen spans was in any way weaker than the others. The stresses produced in the top chords in the standing tests agreed closely with the calculated stresses, but the stresses found in the bottom chords were much smaller than those calculated, averaging only 56 per cent. of the calculated stresses. This is usual in girders of this type, owing to the assistance given by the floor-system to the bottom chords.

It was realized that extensometer tests alone would not ensure that no material was included that had been overstressed and permanently weakened, and this was guarded against during the reconstruction by minute examination of every piece before it was used.

The films used in the Fereday-Palmer stress-recorders were No. 120 Kodak, measuring 2¼ inches by 30 inches. The standing tests needed only an inch or two of the film, so that usually a speed test was also made on the same film. About seventy-five films were used during the two days of testing, and the development of so many films on the spot was rather a problem, especially as the temperature was about 90° F. in the shade. It was particularly desired to develop them on the spot so that any defective tests could be repeated. Some office chuprassis (Indian orderlies) were therefore trained to develop the films; after several lessons and rehearsals they became quite good at the work, and during the actual tests they developed all the films without causing a single failure.

It is now almost 2 years since the bridge was reopened, and no sign of weakness or defect has developed. It may therefore be said that the experiment of using parts of two badly damaged spans to make one good one has been fully justified.

In conclusion, the Authors wish to express their thanks both to Sir James Williamson, the Agent, and to Mr. W. E. G. Bender, C.I.E., M.B.E., M. Inst. C.E., the Chief Engineer of the Bengal and North Western Railway, for permission to present this Paper.

The Paper is accompanied by one sheet of drawings and by seven photographs, from some of which the Figures in the text and the two pages of half-tones have been prepared.

Paper No. 5071.

"The Setting-Out of the Mumbles–Brynmill Section of the Swansea Main Drainage Scheme."

By John Percy Pike, M. Inst. C.E.

(Ordered by the Council to be published with written discussion.) [1]

TABLE OF CONTENTS.

Preliminary.

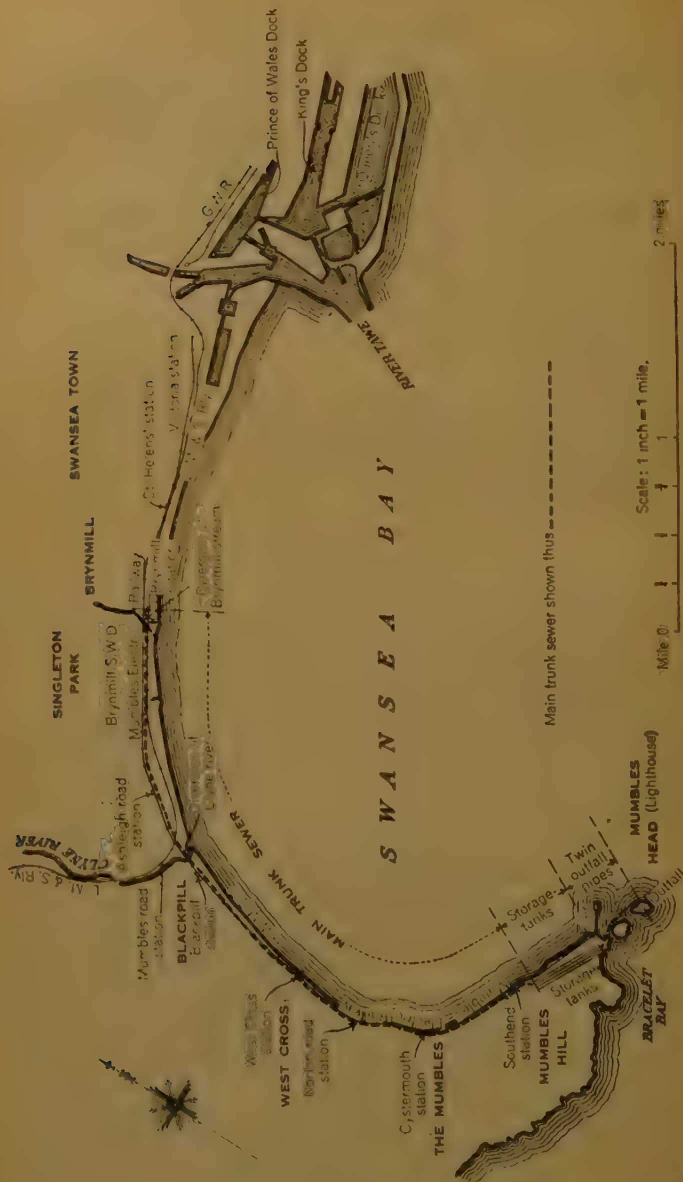
Scope.—The Paper is a description of the setting-out, both before and during construction, of an important portion of the Swansea main drainage scheme. It deals with the main sea outfall, main storage-tanks, screening-chamber and the main trunk sewer from Mumbles to Brynmill. The estimated cost for the complete main drainage scheme was £1,820,000: the portion of the scheme now under consideration was estimated at £653,221 and comprises, briefly, the following works which are shown in *Fig. 1* (p. 58):—

Main Sea Outfall.—This is shown in Figs. 2, Plate 1; it consists of twin 5-foot internal diameter cast-iron turned-and-bored three-lug pipe-lines each 765 yards in length, and extends from the main storage tanks, south of the pier to a point east of the Mumbles lighthouse, crossing the Outer and Inner Sound; 82 yards of the twin pipe-line are in tunnel, whilst the remainder is in open trench on the foreshore. The twin pipes are laid 6 feet 9 inches centre to centre from an invert level of —4·00 O.D. at the tanks to a level of —19·00 O.D. at the point of discharge.

Main Storage-Tanks.—These are four in number and are shown in Figs. 3, Plate 1. They have been tunnelled in limestone rock under

[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—Acting Sec. Inst. C.E.

Fig. 1.



General Plan.

Mumbles hill and are each approximately 2,000 feet long, 16 feet 6 inches wide and 10 feet 6 inches high. They are constructed with 20 feet of rock between adjacent tanks, and the distance from centre to centre of the tanks is 38 feet 9 inches. The total capacity of the four tanks is approximately 7½ million gallons. There is an access-gallery at each end and another placed centrally over the tanks. The top water-level in the tanks is 13·60 O.D.

Screening-Chamber.—The site for the screening-chamber and disintegrating pumps, over which has been built an attendant's house, was prepared by the removal of a portion of the limestone hillside, and is adjacent to the main road. The floor-level of the screen-house is 20·00 O.D. The screening-chamber is at the upstream end of the main storage-tanks.

Main Trunk Sewer.—The principal details of the main trunk sewer are as follows :—

Brick and concrete.

7 feet 9 inches by 5 feet 2 inches and 7 feet 6 inches by 5 feet, ovoid in open cut, to a gradient of 1 in 3,250 : 3,440 linear yards.

7 feet 9 inches by 5 feet 2 inches and 7 feet 6 inches by 5 feet, ovoid in tunnel (mostly "soft" tunnelling), to a gradient of 1 in 3,250 : 750 linear yards.

5 feet 6 inches circular in open cut, to a gradient of 1 in 2,250 : 1,520 linear yards.

Cast-iron pipes on reinforced-concrete piles.

5 feet 6 inches circular, to a gradient of 1 in 2,250 : 530 linear yards.

At Blackpill (*Fig. 1*), the sewer is in siphon under the Clyne river, the siphon consisting of three 3-foot-diameter cast-iron pipes ; near this siphon the sewer crosses under the Mumbles Electric Railway and the London Midland & Scottish Railway. At Brynmill, the sewer again crosses under the Mumbles Electric Railway and then under the main road, where it is constructed over the culverted Brynmill stream, into which there is a storm-overflow from the sewer.

MAIN SEA OUTFALL.

The route chosen for the centre-line of the main sea outfall, from the main storage-tanks to the point of discharge east of the Mumbles lighthouse, included three changes of direction. Intersection-points Nos. 1, 2 and 3, hereafter designated I.P. 1, I.P. 2, and I.P. 3, respectively, and shown in Figs. 2, Plate 1, were established by steel pins set in concrete and centre-punched.

I.P. 2 could not be seen from I.P. 3 owing to a knife-edge of rock between, so a survey-station was fixed on this knife-edge with the theodolite, first approximately when a permanent staging for using

the instrument was set up, and then precisely, when a steel pin on the centre-line was concreted into the rock.

Construction commenced downstream from I.P. 2 at approximately chainage 1,674. A condition was laid down that the radii of the bends in the 5-foot diameter pipes should not be less than 50 feet. The observed angle at I.P. 2 was 145° 15′ 40″, giving an intersection angle of 34° 44′ 20″; but $T = R \tan \frac{1}{2}I$, and, assuming $R = 55$, then T becomes 18·26 feet, whilst if $R = 54$, then T becomes 17·93 feet. The straight pipes commenced approximately 18 feet downstream from I.P. 2, that is, at the tangent-point of a curve of over 50 feet radius.

The master bench-mark for all the works was on a stone wall opposite the yacht club (that is, near the screening-chamber). Reduced levels were taken along the main road and dropped to the foreshore, where temporary bench-marks (steel pins concreted into the rock) were established and the values agreed. The invert of the first pipe laid was − 10·40 O.D. In those cases where some of the level-readings were below O.D. and some above, the Author found it convenient to bring all the levels to a plus quantity for the purpose of making up the level book—deducting again to obtain the reduced level desired.

Pipe-laying proceeded downstream to the hatchboxes (chainage 1,225) and a vertical curve was introduced at the change of gradient (from 1 in 97 to 1 in 300).

The exact calculation for the bends at I.P. 2 was then made. It was as follows (*Fig. 4*) :—Angle PBD = 145° 15′ 40″.

	Measured lengths:		Difference in level:	Calculated level distances:
	feet	inches	feet	feet
AB	20	$0\frac{3}{4}$	7·185	18·729
CB	19	$2\frac{3}{8}$	7·185	17·794
EB	19	$2\frac{7}{8}$	7·175	17·848
FB	20	$1\frac{3}{16}$	7·175	18·774

Then, considering pipe AC for the average distance from B,

(1) The distance of the left-hand side from

$$B = AB \cos 18° \, 16' \, 20'' = 17{\cdot}785 \text{ feet.}$$

(2) The distance of the right-hand side from

$$B = CB \cos 2° \, 51' \, 20'' = 17{\cdot}771 \text{ feet.}$$

Therefore the average distance from B = 17·778 feet.

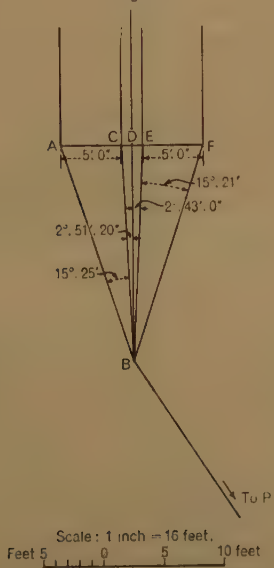
Similarly, the average distance of the pipe EF from B = 17·838 feet.

Also, CD = 0·8865 feet and DE = 0·8478 feet.

Pipe AC.

The length from B, the intersection-point, to the root of the socket $= 17{\cdot}778$ feet; allowing a gap between pipes of $\frac{11}{16}$ inch (this was an average found by practice), and an additional distance of $1\frac{3}{8}$ inch to the point of contact of the pipes, then the length to the tangent-point is $17{\cdot}778$ feet $- (\frac{11}{16} + 1\frac{3}{8})$ inch $= 17{\cdot}606$ feet.

Fig. 4.



Hence, taken on the centre-line of the pipe-track, the radius of the curve $= 17{\cdot}606 \tan 72° \, 37' \, 50'' = 56{\cdot}286$ feet.

Therefore the radius of the pipe-line from AC

$$= (56{\cdot}286 + 2{\cdot}50 + 0{\cdot}8865) \text{ feet} = 59{\cdot}6725 \text{ feet.}$$

Similarly, the radius of the pipe-line from EF = 5313 feet.

The tunnel for the twin outfall-pipes was then driven from the downstream end of the main storage tanks towards I.P. 3 and, at the same time, tunnelling proceeded in an upstream direction under the knife-edge of rock already referred to (that is, from approximately chainage 2045).

A survey was then made in order to ascertain the length of the main sea outfall from I.P. 2 to I.P. 3. The measured line in the triangulation was on the terrace of the pavilion at the pier-entrance.

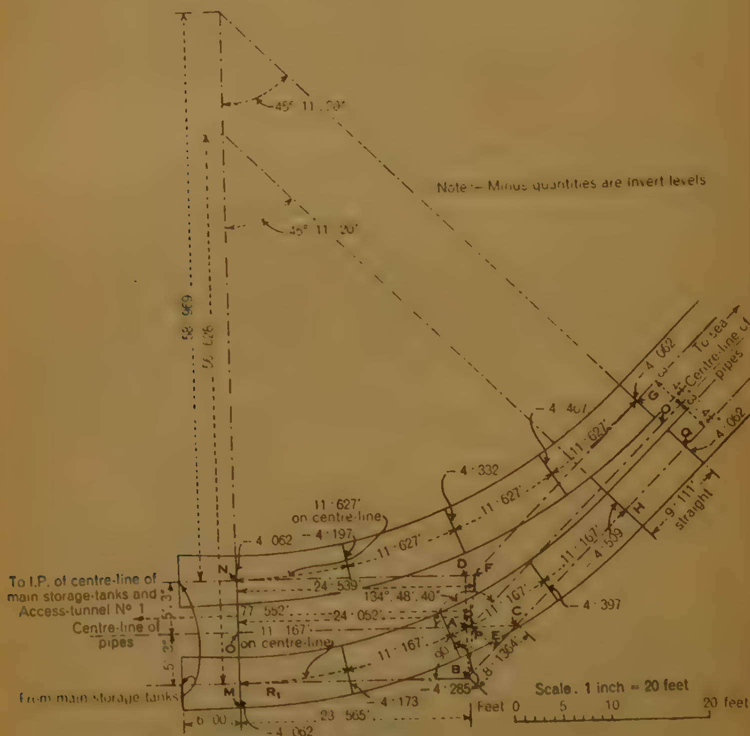
After the bends at I.P. 2 were to hand and fixed, pipe-laying proceeded in an upstream direction also from I.P. 2.

The calculation was then made for the bends at I.P. 3, the centre line of the main storage tanks having been established (*Fig. 5*) :

Angle OPO′ = 134° 48′ 40″, angle ABC = 44° 48′ 40″, and angle BCA = 45° 11′ 20″.

Then AC = 5·25 tan 44° 48′ 40″ = 5·2144 feet. EP = PC sin 45° 11′ 20″.

Fig. 5.



LAYOUT OF BENDS AT I.P. 3.

and therefore PC = EP cosec 45° 11′ 20″

= (3 feet $4\frac{1}{4}$ inches) cosec 45° 11′ 20″

= 4·7273 feet.

Hence AP = 0·4871 feet, DF = 0·9742 foot, NF = 24·5391 feet = FG, and MB = 23·5649 feet = BH.

Whence $R_1 = \text{MB} \tan \dfrac{134° \; 48' \; 40''}{2} = 56{\cdot}628$ feet

and $R_2 = \text{NF} \tan \dfrac{134° \; 48' \; 40''}{2} = 58{\cdot}969$ feet

and HQ = 9·1106 feet.

The pipe-lines between I.P. 2 and I.P. 3 were finally closed by matching-pieces.

The calculation was made for the bends at I.P. 1 in advance. There was a risk in this, as it was almost impossible to keep the joints at each 12-foot pipe abreast (some pipes could be drawn up to give a gap of $\frac{3}{8}$ inch while others were tight at $\frac{7}{8}$ inch), but as the angle was large the consequences were not serious.

The extension of the centre-line was rendered difficult as the middle vertical member of the traveller obstructed observations. Centre-lines for each pipe-line were therefore fixed and extended 6·75 feet apart.

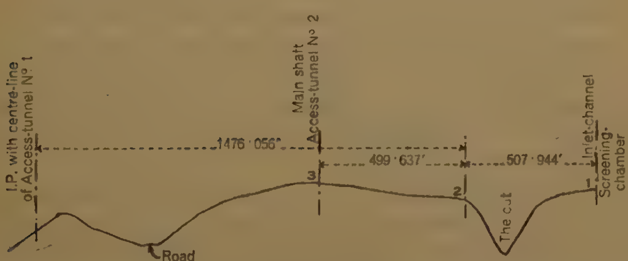
Main Storage-Tanks.

Survey on Surface.

The conditions laid down for the setting-out of the centre-line of the main storage-tanks were as follows :—

At 23·5 feet from an existing survey-station C in the direction of another existing survey-station B, the centre-line at the upstream

Fig. 6.



end of the tanks was required to be 121 feet from, and at right angles to, the line CB. The centre-line at the downstream end was already fixed so far as its lateral position was concerned.

Fig. 6 shows a diagrammatic longitudinal section on the surface of Mumbles hill on the centre-line of the tanks : the station point-numbers agree with the general survey-plan, Fig. 7, Plate 1. The original survey-station C already referred to is also shown in that diagram.

The upstream end of the tanks (point 1) was inaccessible on the surface of the rocky hillside without the aid of quarry harness, and it was impossible to fix a station at this point. Point 1 was approximately 150 feet above the level of the main road. Further up the hillside it was possible to obtain foothold, and a temporary station was fixed by tacheometry on the 121-foot line mentioned above,

but 25 feet beyond the point (No. 1) required—making the "121-foot" line in effect 146 feet.

By a process of trial and error a line was laid down for the extent of the main storage-tanks which was 25 feet south-west of the true centre-line. One of the difficulties encountered in the measurement of the centre-line was in obtaining a sufficient length on a fair surface (the surface of Mumbles hill was very broken indeed) for measured bases in the triangulation. Temporary stations were established at points 1*a*, 2*a*, 3*a*, and 4*a*. Points 2*a*, 3*a*, and 4*a* were then offset 25 feet to the true centre-line, providing points 2, 3, and 4.

A slight error occurred here as the centre-line of the main storage-tanks was not at right angles to the "121-foot" line (the final angle proved to be 83° 11′ 53″). A triangulation was made to obtain the calculated distance between points 1 and 2. Fig. 7, Plate 1, records the general survey-lines used in the setting-out of the main storage-tanks.

Point 2 could be seen from original survey-station C and a measured and corrected line was taken along the main road (shown as 524·011 feet in Fig. 7, Plate 1). Point 2 was then brought in a direct line between point 1 (inaccessible) and the already agreed lateral position of the centre-line at the downstream end (the intersection-point of the centre-line of the main storage-tanks and access-tunnel No. 1, point 4). Point 3 was also rectified. Point 4 was finally adjusted for its true longitudinal position, namely 1,995 feet (the length of the tanks) less 11 feet (the distance from the downstream end of the tanks to the intersection-point).

Steel pins, centre-punched, were concreted in at point 3 and at an intermediate point between points 3 and 4. At point 4 a concrete beacon was constructed for the purposes of visibility, and the theodolite was used on the top of it without a tripod.

At point 2, a trigonometrical station was set up, consisting of a cast-iron pipe concreted in the ground, with facilities for using a theodolite without a tripod. Point 2 is in a prominent position on the escarpment and has been retained permanently at the request of the Officer Commanding Royal Engineers, Western Command.

Owing to the shortness of the bases in calculation for the length of the centre-line of the tanks, a check survey was carried out and a base was measured on the coast road (shown as 911·5 feet in Fig. 7, Plate 1): a staff held vertically over the cast-iron pipe at point 2 could be seen from an existing beacon on the Coastguard hill (Tutt).

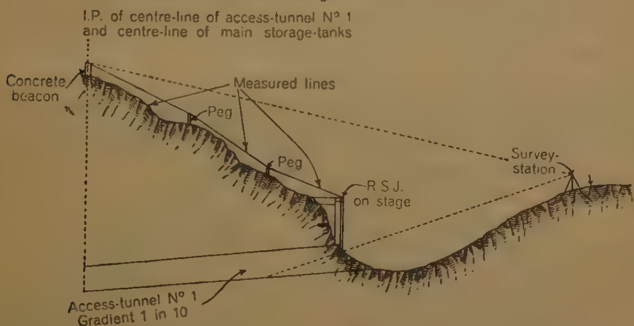
The length and lateral position of the centre-line of the main storage-tanks were thus defined on the surface of the ground, and the upstream end (point 1) was fixed by reference-points.

Access-Tunnel No. 1.

The position of access-tunnel No. 1 is shown in Figs. 2 and 3, Plate 1. The site of the entrance having been decided, a survey-station was set up at the side of the coast road in line with the intersection-point (11 feet upstream from the downstream end of the main storage-tanks) and the centre-line at the tunnel entrance. The intersection-point could not be seen from the entrance. *Fig. 8* shows a diagrammatic longitudinal section.

After blasting had proceeded and the tunnel had been driven some 100 feet a pin was concreted in the floor at the entrance and on the centre-line: the pin also served as a temporary bench-mark, the value being obtained by levelling from the master bench-mark near the screening-chamber.

Fig. 8.



It was then found that the level worked to for the driving was somewhat in error. The level ascribed to an original temporary bench-mark was 0·6 foot in excess of the true value. This meant that from the practical point of view, about 6 inches too much rock had been excavated from the floor for about 50 feet of tunnel. This was rectified by slightly re-grading the tunnel-floor.

For checking purposes and extensions to the centre-line a plummet was suspended from a file mark on a steel joist at the tunnel entrance, the pin already fixed proving unsatisfactory as it became covered in debris; the instrument was fixed under the plummet and the line was continued.

The centre-line was retained by centre-nails in wood plugs fixed in high points in the roof on account of the blasting: these nails were also used for levelling purposes. An inverted levelling staff was used, the back-sights being subtracted from the reduced level to form the collimation-value whilst the intermediates and fore-

sights were added to the collimation-value to obtain the new reduced levels. It was necessary that the length of this tunnel should be carefully measured and the intersection-angle correctly transferred below, as the access-gallery (at the end of the tunnel and at right angles to the main storage-tanks and above these tanks) was to be driven from access-tunnel No. 1.

The method of arriving at the length of access-tunnel No. 1 from the tunnel-entrance to the intersection-point (shown in *Fig. 8*) was to drive wooden pegs at intervals on the centre-line on the surface of the ground and to fix centre-nails; the comparative levels of the heads of the nails were observed and the slopes were measured; each triangle (each slope being the hypotenuse of a right-angled triangle) was then solved to obtain the horizontal distance; the horizontal distance was once more translated into the equivalent length for a slope of 1 in 10 and the position of the intersection-point fixed underground; the same intersection-angle observed on the surface was transferred below and was also fixed with a peg and a reference-peg.

While access-tunnel No. 1 was being set out a slight discrepancy in the position of an existing brick building from that shown on the Ordnance map was discovered; checks and re-checks were made before such a rare conclusion was reached.

Access-Tunnel No. 2 and Main Shaft.

This tunnel led from the main road to the access-gallery approximately mid-way along and over the main storage-tanks. The gradient was 1 in 7. The tunnel is shown in Figs. 3, Plate 1, and a diagrammatic longitudinal section of access-tunnel No. 2 and a cross-section of the main storage-tanks is shown in *Fig. 9*.

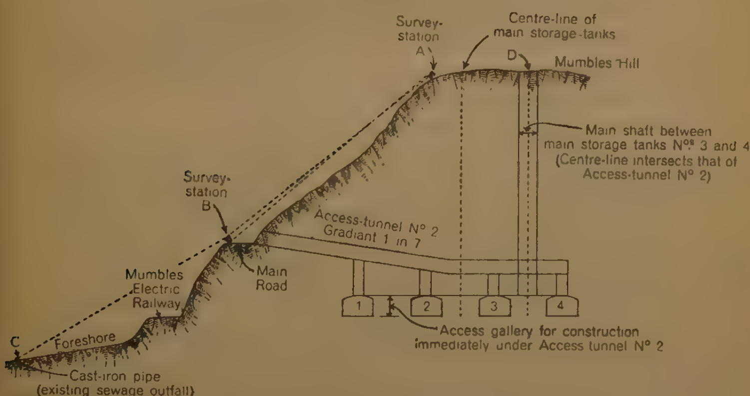
A convenient take-off for the tunnel from the road approximately mid-way with respect to the length of the main storage-tanks was chosen, and a beacon (consisting of a steel pin concreted in and centre-punched) was fixed at right angles to the centre-line of the tanks and was marked " A " on *Fig. 9*. The line " A " to the centre-line of the tanks was continued in either direction, towards Swansea Bay with a pin in the road at " B," and on to the foreshore (where it was useful at low tide) to an existing cast-iron sewage-outfall, " C "; and also in the reverse direction, marking the centre of the main shaft " D ".

Operations commenced on the main shaft and the adit (access-tunnel No. 2) simultaneously. During the earlier period of the shaft-sinking, when a compressed-air hoist was used, it was possible to employ a central plummet in the shaft. Later, however, headgear

with an electric hoist was installed, and the shaft plummet had to be off-centre ; this somewhat complicated the observations.

It was not possible to see the beacon A from B (*Fig. 9*), so a tripod with a plummet was erected at A and observations were taken on to the plummet-line from B ; owing to the frequent windy weather causing the plummet to swing, it was the practice to shield the line from the wind whilst bringing it over the station, and then to tie it in position. Observing at B, A was sighted and the instrument was reversed on to C. Extensions were then made to the centre-line of the tunnel from the road until visibility was cut off : the centre-line was continued from plummet-lines in the tunnel. In this tunnel, holes were drilled in the roof for wood plugs in duplicate, steel

Fig. 9.



dogs were fixed, and the centre-line was defined by file marks on the dogs.

Levelling proceeded as before from the master bench-mark near the screening-chamber, and again it was convenient to use an inverted staff. Driving was continued in the tunnel until it covered tanks Nos. 1 and 2. The main shaft was continued to the floor of the tanks.

A connecting construction-tunnel at right angles to the main storage-tanks was then driven from the main shaft up to and including tanks Nos. 1 and 2 at the floor-level of the main tanks. Galleries were then extended upward from Tanks Nos. 1 and 2 to access-tunnel No. 2.

It was then possible to apply a check. Plummets were suspended on the centre-line of access-tunnel No. 2 as far as tanks Nos. 1 and 2, and this line was continued to the centre-line of the shaft. From

the centre-line thus obtained, the whole of the driving of the tanks was set out; driving was carried out both ways and it was possible to work on eight faces of the tanks.

MAIN STORAGE-TANKS: CONTINUATION OF SETTING-OUT.

Owing to the importance of the base-line on the centre-line of access-tunnel No. 2 from tank No. 1 to tank No. 2, the work was checked and re-checked.

It was essential in the early stages to do all observations on a Sunday, as three shifts were being worked during the week in tunnelling-operations.

Heavy plummets suspended by steel wires were used in tanks Nos. 1 and 2, and the weights were placed in buckets of water to steady them. Even with this precaution it was impossible to prevent the twisting action, and steel wire has the disability that it becomes distorted and allowance has to be made in the observations for this; these observations of necessity had to be carried out slowly.

The method adopted of extending the base-line was to set up the theodolite between tanks Nos. 2 and 3 and by trial and error to perform the laborious operation of fixing the instrument in line with the two plummet-wires. For sighting the plummet-wires, electric torches with large reflectors were held with the light towards the instrument but slightly tilted to avoid glare.

The access-tunnel No. 2 was measured on the slope, the horizontal distance was calculated (*Fig. 10*), and pegs locating the positions of the centre-lines of all the tanks were fixed. The measurements giving the lateral distances between centre-lines of the tanks were then taken exactly with a steel tape from the wire of the nearest heavy plummet to a small plummet-wire suspended from a tripod, and on to a pencil-mark on a rail; measurements were then extended from the pencil-mark to common pins stuck into boards weighted with rails. Occasionally a temporary station of this kind became inadvertently displaced, but with the employment of experienced chainmen such accidents were reduced to a minimum. The instrument was then set up over each of the temporary stations (which were the intersection-points of the central communication gallery and the four tanks, and in the same straight line as the centre-line of access-tunnel No. 2) and two dogs were fixed in the roof of each tunnel with file-marks indicating the centre-lines. In several cases the dogs were blown away, but a sufficient number was retained to allow the lines to be produced in each tunnel. The extension of the centre-lines also took place on Sundays.

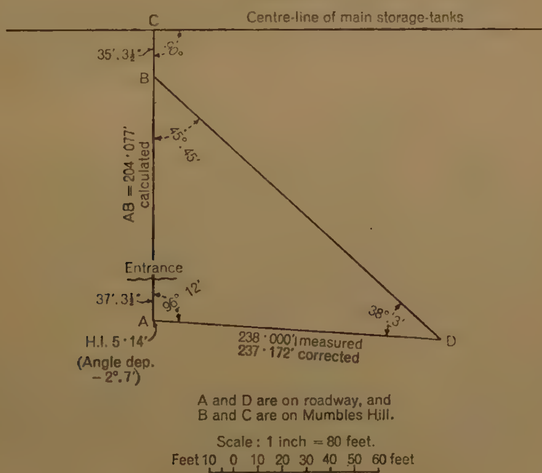
When driving was well advanced it was possible to use instruments

in the tanks while work was proceeding, but the reverberations due to blasting and the operation of the tip-trucks, mechanical loaders, etc., made observations very difficult and increased the risk of error.

Driving proceeded upsteam at a greater rate than in the downstream section, the rock downstream from access-tunnel No. 2 and the central access-gallery proving more fissured than in the upstream section.

The driving of tank-tunnel No. 1 was expedited on account of the desirability of a check from the upstream end, and in due course communication was established with the screening-chamber excavation. As soon as the excavation at the screening-chamber enabled the perforation to be made in the rock and communication was

Fig. 10.



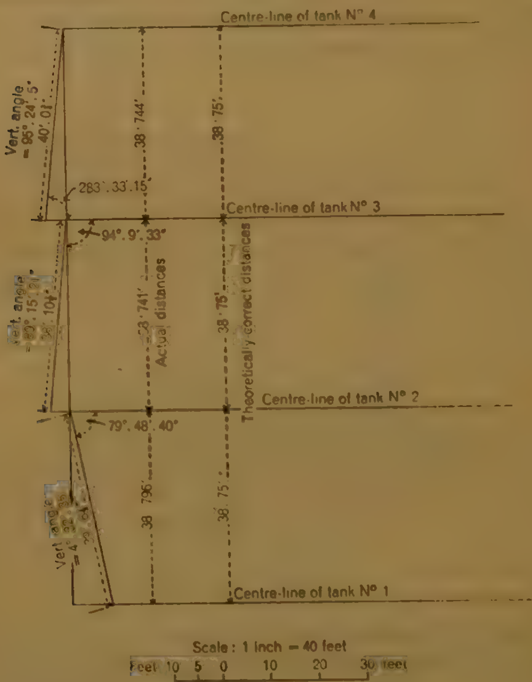
established at the upstream end, the inlet-channel and access-gallery were driven, enabling point No. 1 (shown in Fig. 7, Plate 1) to be fixed underground. Fig. 7, Plate 1, records the survey and the relation of the centre line of the inlet-channel to the original survey-station C.

The downstream termination of the " 1995 " line was fixed underground as follows: Access-tunnel No. 1 having already been driven as described, it was continued through the intersection-angle observed on the surface for a distance of 11 feet from the intersection-point, the distance of 11 feet being in the same straight line as the centre-line of the tanks. The downstream access-gallery at right angles to the 11 feet of tunnel was then driven (the access-galleries in each case being over the top of the tanks), and the downstream centre-lines

of the four tanks were set out in this access-gallery. As soon as the main storage-tank tunnels were driven through to the downstream end, a perforation was made upwards into the access-gallery, allowing the centre-lines to be fixed in the four tanks below. *Fig. 11* shows the check made on the centre-lines of the four main storage-tanks at the downstream end.

The accuracy of the survey for the length of the tanks is shown by the fact that the theoretical length of the tanks was 1,995 feet,

Fig. 11.



and the actual measured length after completion was 1,994 feet $10\frac{13}{16}$ inch.

The levels were obtained from the master bench-mark near the Yacht club, and were transferred for driving purposes through access-tunnel No. 2. For the lining of the tanks levels were taken through the screening-chamber excavation into the four tanks, and temporary bench-marks (steel pins in concrete) were set in the tank-floors at convenient lengths. Instead of using the levelling staff a steel tape (with feet divided into tenths) was used for transferring levels into the screening-chamber excavation (the excavation was about 25 feet

deep), the tape was plumbed, and the reading was taken direct on to the tape.

In connexion with the extension of the centre-lines in the tanks, an illuminated plummet was devised for use as a target. Used as a plummet there was a slight risk of error due to the refill-batteries not being of the same specific gravity throughout their substance; for blasting lines, however, its use was quite successful. For final precise work the illuminated apex of the cone was held in position by the chainman with the point on the file-mark of the dogs fixed in the roof of the tanks. The visibility of the plummet depended on the amount of smoke present from blasting, but with a clear atmosphere the cone could be seen through the instrument at a distance of 500 feet. For the complete half distance of the tanks (approximately 1,000 feet), a flood light with cross-tapes set in position was used as a target.

There was no discrepancy between the design and the finished tanks in lateral position or in levels.

Screening-Chamber.

The preparation of the screening-chamber site involved the removal of about 6,000 cubic yards of the limestone rock forming Mumbles hill from a position adjacent to the main road. The lamination in the rock proved to be at approximately 60 degrees to the horizontal, and the rock was cleared to this angle for the length required.

As a protection to the traffic on the main road and on the Mumbles Electric Railway, a large portion of rock was left standing adjacent to the main road while blasting operations proceeded behind it. Two holes, sufficiently large to take motor-lorries, were made through this mass of rock to give communication with the main road. Survey-lines for calculating the centre-line of the screening-chamber and the centre-line of the inlet-channel to the main storage-tanks were taken through these holes from the survey-line CB referred to on p. 63. Calculations for the amount of rock to be taken from the hillside and also for the quantities of rock excavated were carried out by tacheometry from the line CB.

Main Trunk Sewer.

The greater part of the length of the main trunk sewer was in close proximity to the Mumbles Electric Railway, and most of it was also near the main road. The section between Blackpill and Swansea Bay (*Fig. 1*, p. 58) had, in addition, the L. M. & S. Rly. on the Swansea Bay side. The vibration set up by the various lines

of traffic made observations with both theodolite and level difficult: it also occasionally affected the values of temporary bench-marks, consisting of steel pins set in concrete on the surface of the ground.

One of the first operations in the setting-out was to fix temporary bench-marks; these were established approximately every 300 feet from the master bench-mark near the screening-chamber. Owing to the three traffic-lines being in close proximity at Singleton Park, the temporary bench-marks were fixed on the further side of the main road from the sewer, as shown in Fig. 12, Plate 1.

The projection in the toothing of the invert of the brick lining of the sewer as it proceeded was sometimes used as a temporary bench-mark, but the most convenient level-reference below ground was found to be a steel pin set in concrete.

The centre-lines for the sewer were projected when required from the 1/1,250 working drawings. At every change of direction a curve was set out. The curves range for the most part between 500 feet and 1,000 feet radius except at the two main road crossings at the screening-chamber and Brynmill, which are 50 feet and 90 feet (approximately) respectively. It was occasionally necessary to introduce a transition curve or to re-align a curve, and in order to determine the offsets more accurately, a diagram was used in which the chord-scale was a multiple of the radius-scale, and the offset-scale was the same multiple of the chord-scale.

The main trunk sewer was connected with the screening-chamber by a road-crossing in tunnel. By the time the construction of the sewer reached this junction the screening-chamber site was available for direct-observation purposes, as the rock had been excavated to the invert-level of the sewer.

An existing sewage-storage tank was in such close proximity to the new sewer that it was deemed unwise to set out the centre-line before gaining information by the means of trial-holes of the thickness of the existing construction; the trial-holes were therefore sunk at the nearest points of contact.

One of the sections of the sewer in tunnel between West Cross and Blackpill stations (*Fig. 1*) was constructed under buildings. There was no visibility on the centre-line, and the main road was on one side of the sewer and the Mumbles Electric Railway on the other. There was a change of direction mid-way in the tunnelled length (at M.H. 21, which was used as a shaft). The centre-line was set out by means of offsets into and along the main road, which carried considerable traffic. Elaborate precautions were taken when the instrument was set up in the carriageway, a barricade being erected and a flagman being employed.

The culverting in reinforced concrete of the Clyne river and the

construction of the main trunk-sewer siphon beneath it were carried out early in the course of the works ; very great care was taken with the levels, as the sewer on the Swansea side of the L. M. & S. Rly. had already been completed. The construction of the sewer was proceeding at several points at the same time and it was essential that the same gradient be maintained. Fortunately it can be recorded that, except in one short section, the invert-levels of the sewer were in accordance with the gradients shown on the drawings : the short section referred to (about 50 feet) was cut out and replaced.

The portion of the main trunk sewer under the L. M. & S. Rly. at Clyne river was carried out in cast-iron tubbing. For the establishment of the centre-line the theodolite was set up on the railway embankment, and by the process of trial and error a station was fixed at the rail-level in line with stations at ground-level on either side of the embankment. Two centre-nails were driven on the struts of the shaft on the Mumbles side of the railway (the heading being driven from this side) in the same straight line.

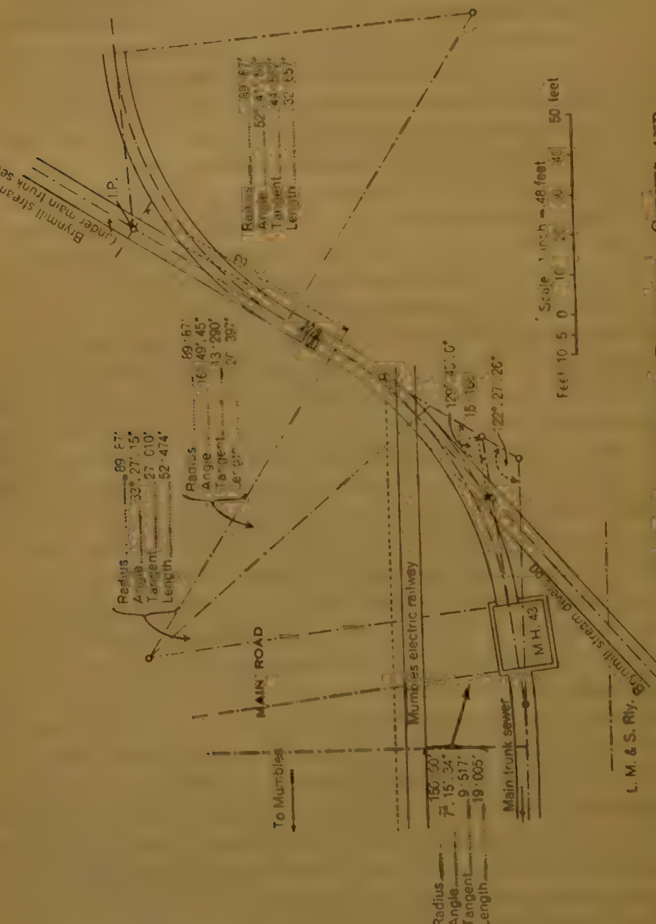
The first curve to be set out was on the section known as the Burrows, between Blackpill and Ashleigh Road (*Fig. 1*). Wood pegs with centre-nails were fixed in the sandy soil ; as excavation proceeded, the pegs were allowed to be dug up, and as no reference-points were made the setting-out of the curve had to be done again. The theodolite was set up over the tangent-point and was supported by heavy timbers across the 8-foot trench to avoid movement. Special struts were fixed at the chord-lengths, and the deflection-angles were read and indicated on the struts by centre-nails.

At approximately 530 yards downstream from Brynmill it was decided to adopt a reinforced-concrete foundation for the main trunk sewer, and 5-foot 6-inch cast-iron pipes were substituted for the brick-and-concrete construction. The requirements of the specification were that the piles should be driven to a set of not more than 1½ inch for the last ten blows of the hammer, which weighed 2 tons and had a drop of 2 feet 6 inches. It was apparent, without taking minute observations, when the piles began to take this set, and the method then adopted was to use a dumpy level and a levelling staff, the base of which was held on a pencil line marked on the concrete pile. The level was usually set up on the other side of the main road on account of the vibrations due to the pile-driving.

At Brynmill the Brynmill stream had to be culverted and the main trunk sewer and the stream taken across the main road in one trench, the Brynmill stream being below the sewer. There was a storm-water overflow at the adjacent manhole (No. 43) into the Brynmill stream culvert. The problem here was to set out a curve

between the general line of the sewer parallel with the Mumbles Electric Railway and the centre-line across the main road, whilst allowing for a piece of straight at manhole No. 43. *Fig. 13* shows the general arrangement of the curves and lay-out at Brynmill. The road crossing was undertaken in trench (a portion of the road

Fig. 13.



MAIN TRUNK SEWER AND BRYNMILL-STREAM DIVERSION: CURVES AND ROAD-CROSSING AT BRYNMILL.

being closed at a time), so most of the observations could be made on the surface.

Some difficulty was experienced in retaining the intersection-point near the Mumbles Electric Railway on account of the railway crossing. Observations for the centre-line of the Brynmill stream diversion were carried to a point on the sea-wall on the Swansea Bay

side of the L. M. & S. Rly., and were then transferred to the foreshore-level.

Observations and Instruments.

A Cooke, Troughton & Simms "Tavistock" theodolite was used throughout the work, with which the reading obtained is the result of simultaneous setting and observation of two circle-divisions situated 180 degrees apart. The circles are well illuminated by daylight for surface work and by electric lamps (in the instrument) for underground observations; the diaphragm is also illuminated.

For further facility underground it would have been a convenience if the spirit-levels could have been lighted; illumination for these was given by electric torches. A screw adjustment-device for moving the instrument at right angles to the line of observation would also have been very useful for lining up with two targets. Important readings for line-extensions were always taken by turning through 180 degrees and by repetition, and never by reversing the telescope.

It was possible with this instrument to read to a single second; although it is probable, even by repetition, that no two readings would agree to a single second, the instrument was always read to the limit, and the method was on the side of accuracy. The instrument was in the charge of one engineer, and it was always used by him as a precaution against accidents.

The levels used were of the modern type where the line of collimation remains constant throughout the range of focus of the telescope. The position of the spirit-bubble was reflected in an inclined mirror. Owing to the accuracy of the levelling which was required for gradients of 1 in 2,250 and 1 in 3,250, levels were read to three places of decimals, the third being approximate.

Some misunderstanding resulted in checking by reason of one set of readings having been taken from the inclined mirror and another set direct from the spirit-level; there is a slight variation between these two readings, and the readings should always be taken from the mirror. To eliminate such variations a further level was obtained fitted with a prism-reader by means of which coincidence of the reflected images of half the ends of the bubble of the spirit-level indicates that the telescope is truly horizontal. This instrument was used in all precise work.

For work underground with the level two hand-torches usually sufficed, one being used on the staff and one on the instrument, It is, however, very convenient for the engineer in charge of the level to have the spirit bubble illuminated, as he can then use a hand-torch to light the diaphragm, and need not change his position.

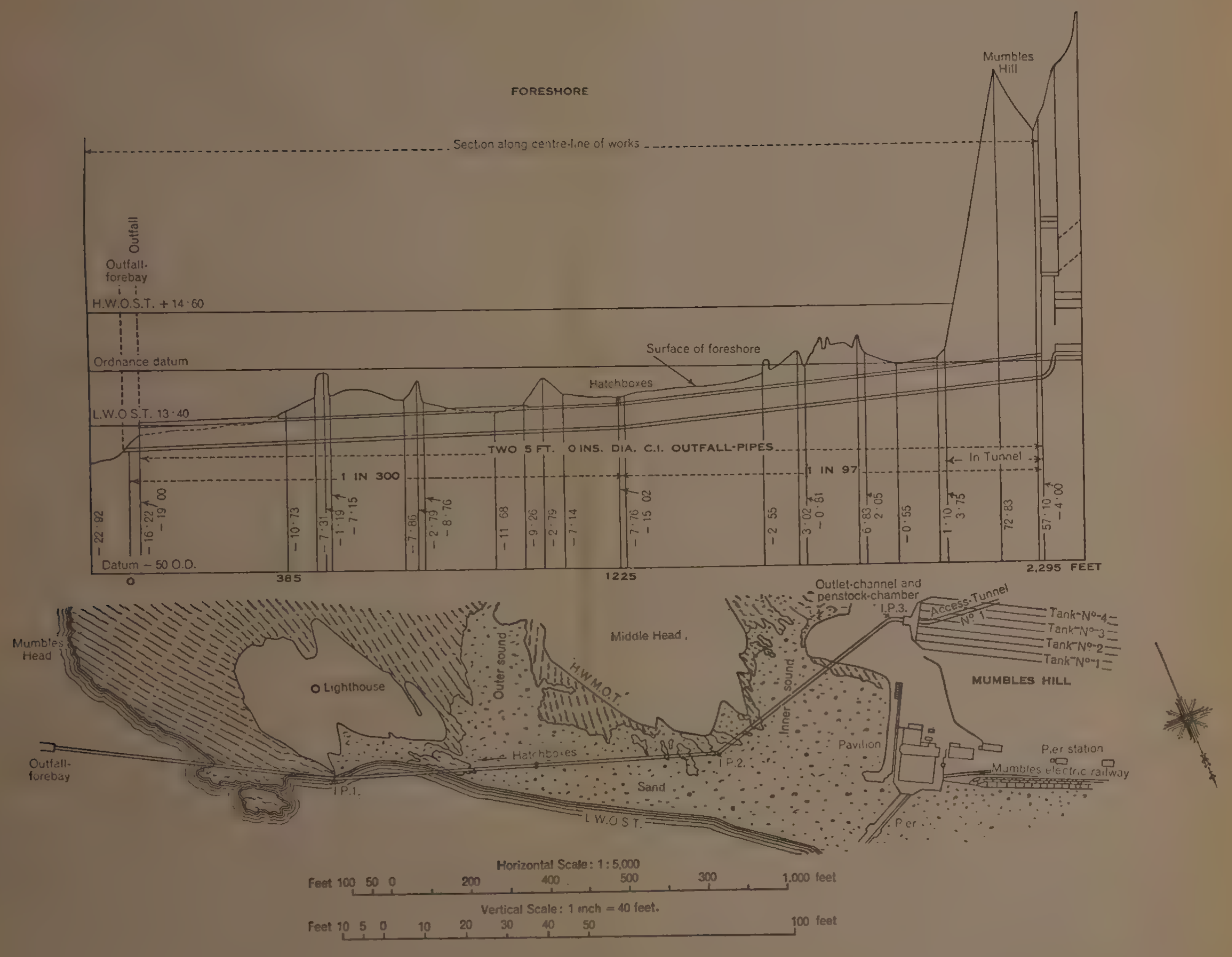
Acknowledgements.

The Author's acknowledgements and thanks are due to the following for the supply of, and permission to use, information in connexion with the preparation of this Paper: Mr. J. R. Heath, Chief Engineer, Swansea Main Drainage Scheme, and Messrs. S. S. Harrison, James McBride, B.Sc., Assoc. MM. Inst. C.E., N. Steele and T. O. Walters; also to Messrs. Cooke, Troughton & Simms, Ltd., and Messrs. Stanton Ironworks, Ltd.

The Paper is accompanied by eighteen sheets of drawings, from some of which Plate 1 and the Figures in the text have been prepared, and by five photographs.

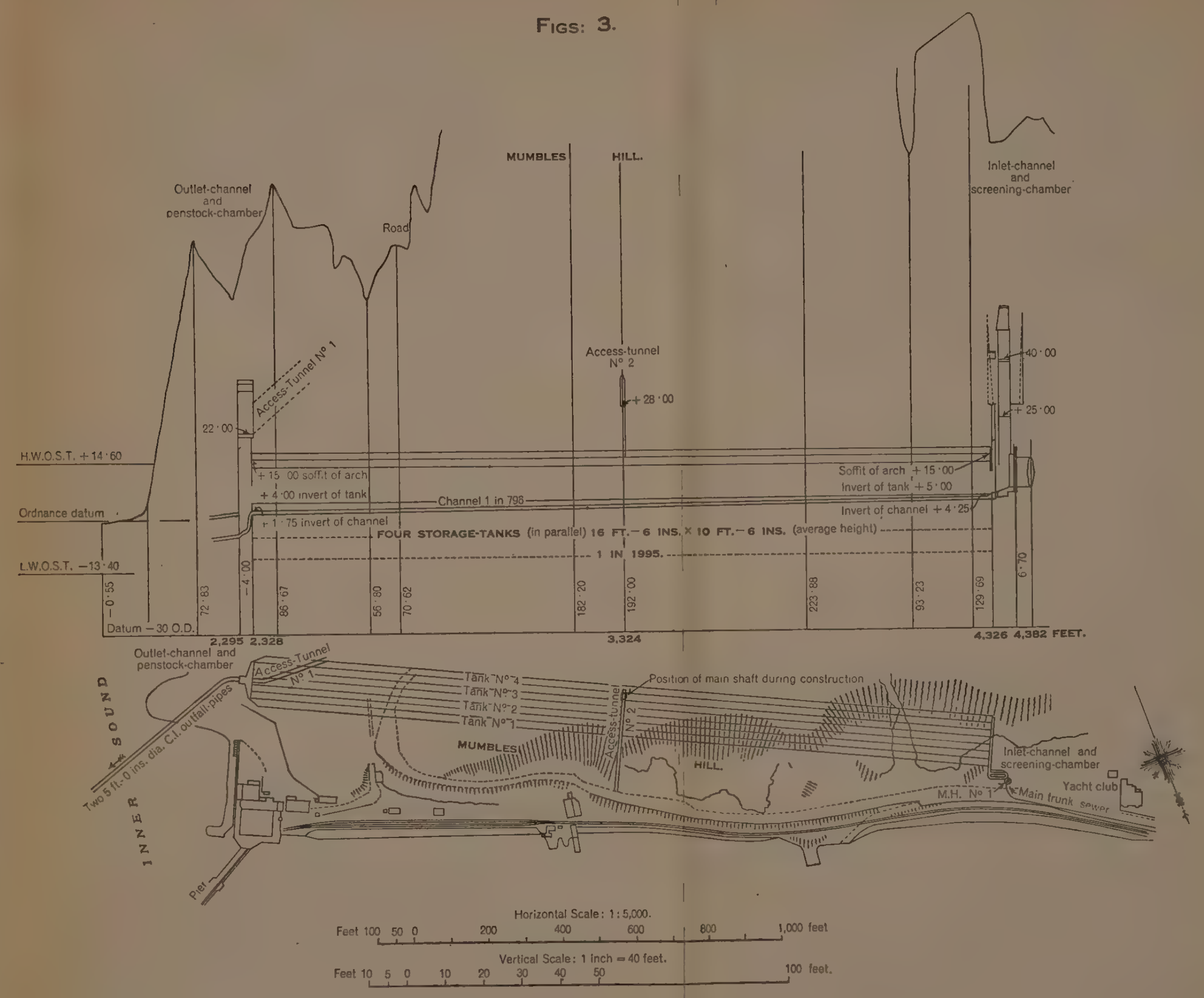
THE SETTING-OUT OF THE MUMBLES—BRYNMILL SECTION OF THE SWANSEA MAIN DRAINAGE SCHEME.

Figs: 2.



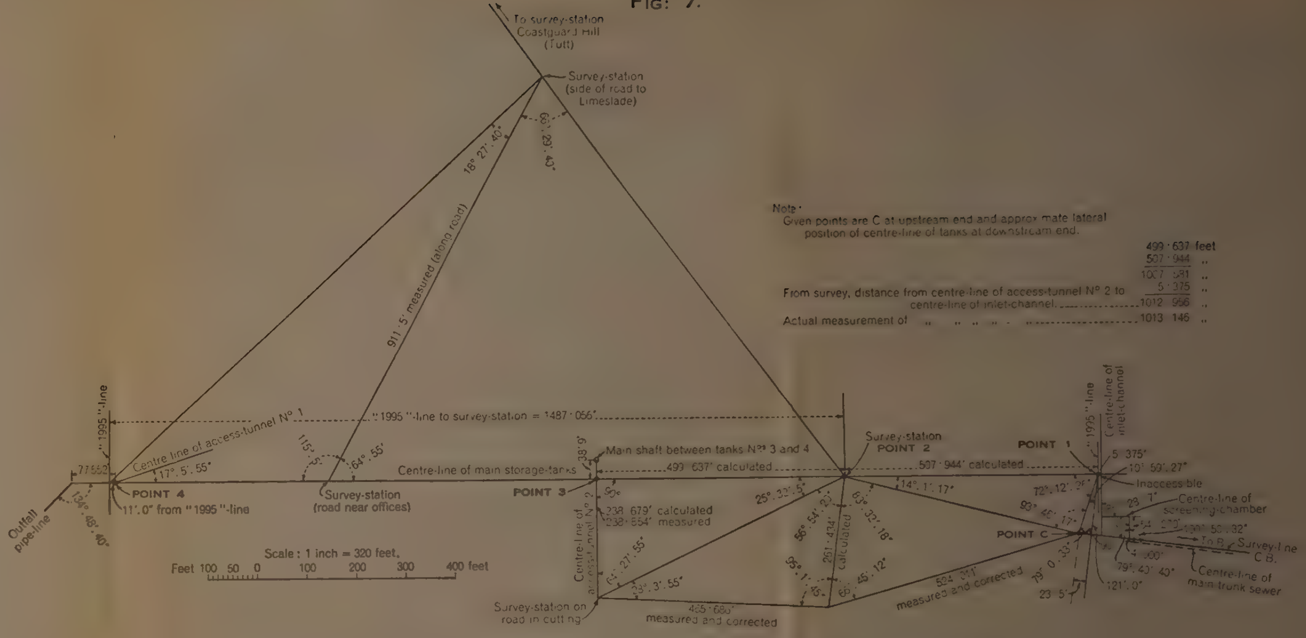
SECTION AND PLAN OF MAIN SEA OUTFALL.

Figs: 3.



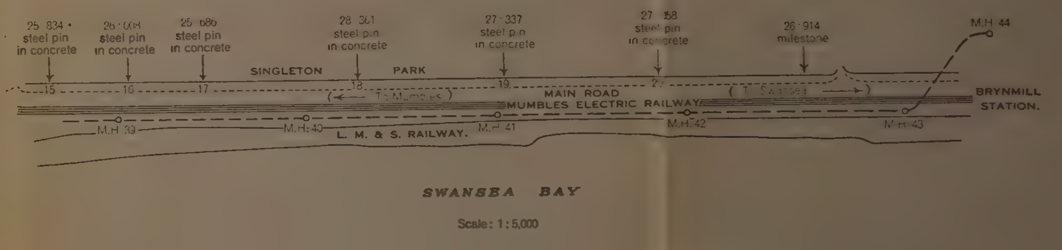
SECTION AND PLAN OF MAIN STORAGE-TANKS.

Fig: 7.



GENERAL SURVEY FOR CENTRE-LINE AND LENGTH OF MAIN STORAGE-TANKS.

Fig: 12.



TEMPORARY BENCH-MARKS BETWEEN MANHOLES Nos 39 AND 44.

J. P. PIKE.

William Clowes & Sons, Limited: London.

Students' Paper No. 937.

"Modern Swimming-Pool Design."

By EDWIN LOMAX, Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

INTRODUCTION.

A MODERN swimming-pool should, if possible, be designed as one unit of a complete scheme of sport and recreation, which might include many, if not all, of the following features :—

A winter (covered) swimming-pool ;
A summer (open-air) swimming-pool ;
Sunbathing beaches, high-diving pool ;
Gymnasium, tennis-courts, putting-greens, running-track and playing-fields ;

together with the necessary changing-rooms, refreshment-rooms, laundry, car-park and administration-buildings.

The conservative English attitude towards sports-stadiums and recreation-centres is only just awakening to the necessity and advantages of such schemes. If the opportunity to design such a comprehensive scheme does not present itself, however, its principles at least can be embodied in the design by providing for physical exercise, unlimited sunlight, both natural and artificial, and a fresh, natural atmosphere. Recently-developed methods and materials give the engineer scope for unconventional treatment and make for

[1] Correspondence on this Paper can be accepted up to the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—ACTING SEC. INST. C.E.

almost ideal conditions. Reinforced and lightweight concrete, high-tensile and stainless steels, and glass in its many and varied forms all further the interests of utility and amenity in the construction of a swimming-pool.

The modern swimming-pool must try to provide ideal conditions for swimming and recreation in healthy, safe, and attractive surroundings.

DESIGN.

Lay-out.—Where, as suggested, swimming-pools form part of a comprehensive scheme of sport and recreation, the main buildings must be arranged on the north and east sides of the open pool, in order to protect bathers and the water from cold winds, and at the same time to leave the south and west aspects open to the sunlight. For the same reasons, the winter bath should be arranged so that its sliding doors and windows face the south and west.

Planning.—The keynote of modern planning is to separate the bathers from the spectators, primarily on hygienic grounds. The bather must be made to follow a definite circulation—cash-desk, dressing-hall, and finally showers and footbaths—before entering the main swimming-bath. From this point he may proceed to either the winter bath or the open-air pool, as he may desire or weather conditions permit. As mixed bathing is almost universal, it is usual to arrange that the dressing and sanitary accommodation is duplicated on opposite sides of the building. Spectators proceed from the cash-desk by special corridors to the spectators' balconies of the winter bath, or the spectators' terraces of the open-air pool.

The administration and service buildings must be placed centrally, so as to be in touch with all activities, and also for economical planning, maintenance and management. Thus, the laundry, filter-house and boiler-house should all be in close proximity, and also the superintendent's office, café, committee-room, store-room, ticket-office and attendants' room could all be planned together with advantage.

The exits from the baths may, in normal times, be merely returns through the entrances, but emergency exits must be provided for special occasions such as galas and carnivals, when large numbers of spectators will be leaving the building at the same time.

Shape of Pool.—The planning of the actual pool presents a problem in itself, as there are so many conflicting advantages and dis-

advantages in the various shapes and contours which are possible. The following facts and considerations usually affect the design for any particular case.

In accordance with the regulations of the Amateur Swimming Association, the swimming area must be rectangular if it is to be used for competitive events, and the following data must be considered.

Championship races exceeding 500 yards are not permitted in pools less than 165 feet in length; this may therefore be assumed to be a standard and economic length for open-air pools, although many large pools are 330 feet in length. The width of the open-air pool may vary, but should not be less than 60 feet, which is the maximum width of the field of play in water polo. For the winter swimming bath the standard economic length which is now recognized is 100 feet (in any case, it should not be less than 75 feet). The recommended width is 48 feet, but if this cannot be secured the width should in no case be less than 27 feet, which is the minimum width for a final in the game of water polo as controlled by the Association.

The economic area of the pool can be computed by estimating a maximum attendance and allowing 20 square feet per bather, 50 square feet per diver, and assuming 80 per cent. of the bathers to be in an area less than 4 feet 6 inches deep. It is usual during rush periods for probably only 60 per cent. of the bathers to be in the water at the same time.

With the above regulation sizes in mind, the shape of the pool can be designed.

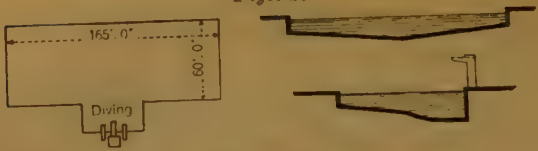
The plain rectangular pool (*Figs. 1*) is probably the commonest form, but it has the disadvantage that no separate area is provided for diving; thus a person desiring to swim from end to end of the pool runs the risk of injury, or at least interference, by the divers.

Figs. 1.

It is assumed that the rectangular shape is simpler of construction and more economical in planning, but with the greater use of reinforced concrete for pool and buildings the advantage, if any, is very slight.

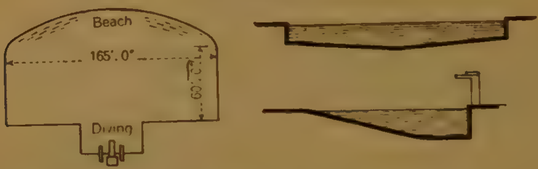
The rectangular pool shown in *Figs. 2*, which provides for a separate "diving-pool" off the main swimming line, is a definite improvement.

Figs. 2.



Figs. 3 shows a shape which seems to approach the ideal for the

Figs. 3.



summer pool, combining a beach for children and learners, a rectangular swimming pool, and a separate pool for diving.

The type shown in *Figs. 4*, combining the rectangle and circle,

Figs. 4.

has many desirable features, particularly for the winter pool. It has the disadvantage that no separate diving-pool is provided.

The semicircular-ended type (*Figs. 5*), although it does not comply

Figs. 5.

with the regulations, has been used in small private pools, as it lends itself to decorative treatment.

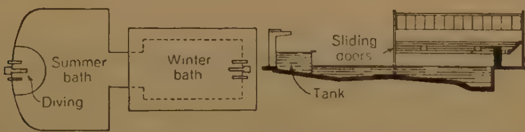
The same remarks apply to the design shown in *Figs. 6* as to the

Figs. 6.

previous type, although it can be used in quite large and, preferably, open pools. The advantages are that the diver is not obstructed by the non-swimmer, and the non-swimmer cannot fall into deep water from the side. Overcrowding is also reduced.

In the opinion of the Author the ideal design is one which combines the open-air pool and the covered pool, as shown in *Figs. 7.* In winter the pools could be separated by means of a

Figs. 7.



"lock-gate," and the double sliding doors could be closed. In summer, the two pools would be combined, and the bather could choose either bath as his fancy or the weather dictated.

Weather Protection.—The summer bath must be well protected from the wind, preferably by means of the tiers of the spectators' seating accommodation. Trees and shrubs may be planted upon the top of an artificial embankment, forming a picturesque background, but they must not be placed too close to the pool, on account of the nuisance caused by falling leaves. Glass screens might also be employed for protection from the wind, fixed from 10 feet to 15 feet from the edge of the pool. These screens would also provide an effective barrier for separating the bathers from the spectators, whilst still allowing a full view of the pool.

Provision should also be made for sunbathing by means of shingle beds, which should be adequately underdrained. If a coloured shingle is chosen, it will absorb the sun's rays far better than a white shingle, besides giving a more restful appearance. On an irregular site an otherwise wasted space can often be utilized as a sunbathing beach.

Contours of Pool.—The contours of the floor of the pool necessitate as careful consideration as the shape and size of the pool. It is usual to make 75 per cent. of the area of the pool less than 4 feet 6 inches deep, in order to avoid overcrowding of the learners and non-swimmers, forming the majority of bathers, besides making the pool safer. Sharp changes of gradient must be avoided until a greater depth than, say, 6 feet has been attained. At a shallower depth than 6 feet the floor should not slope more than 1 in 16, and a maximum slope of 45 degrees is usual at a greater depth than

6 feet. The aim of the designer should be to secure the easiest gradients that can possibly be obtained under the circumstances. The maximum depth for children and learners may be taken as 3 feet 3 inches, but if a "beach" can be incorporated it inspires much greater confidence and enjoyment. For water polo, a minimum depth of 6 feet is desirable, and, although the minimum depth, under the rules, is only 3 feet, it is impossible to play the game correctly at a less depth than 4 feet 6 inches.

Diving Boards and Pool.—The dimensions and depths of water required are controlled by international standards. It may be assumed as a general rule, however, that the depth of water need not exceed half the height of the diving board, with a minimum of 9 feet and a maximum of 15 feet. This maximum depth should be immediately below that running board which projects furthest over the water, and it should be carried forward as necessary to suit the height and character of the board provided for. Thus a 3-metre spring board requires a water-depth of 9 feet to 12 feet, carried 15 feet forward; a 5-metre board requires 10 feet to 12 feet depth of water carried 20 feet forward; and a 10-metre board requires 15 feet depth of water carried 25 feet to 30 feet forward.

The diving boards usually provided are a 1-metre running spring board, 16 feet long by 1 foot 8 inches wide, and a 3-metre running spring board, 16 feet long by 1 foot 8 inches wide.

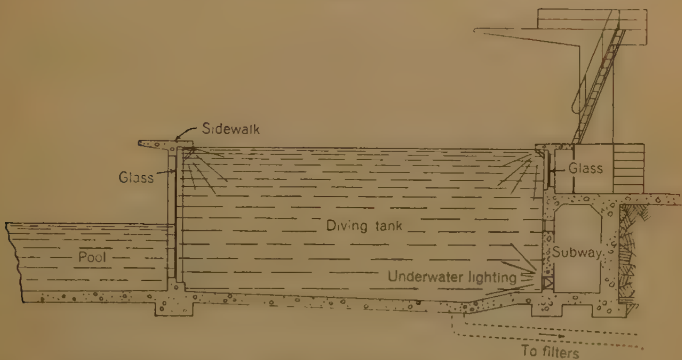
The Author considers that a spectacular form of diving tank, or pool, could be constructed of armour-plate glass in a reinforced concrete frame (approximate coefficients of expansion per °F. are: glass, 5×10^{-6}; concrete and stainless steel, $7 \cdot 4 \times 10^{-6}$). Combined with underwater lighting, such a scheme possesses great possibilities for entertainment, and also for instruction in diving and swimming. A suggested design is shown in *Fig. 8*.

Construction.

General Considerations.—If the foundation does not possess good natural drainage it will be necessary to provide an adequate system of land drains to a sump, or a number of different sumps. By the latter method it would be possible in the unfortunate event of leakage to locate the fault with greater accuracy and so to expedite repair work. In certain cases the opposite effect of external water-pressure will be encountered, and the pool must then be designed as a "tank." It would be advisable to provide relief-pipes for the external water-pressure, in order to avoid the lifting of the floor when the pool is emptied for cleaning or repair.

The usual type of construction, however, is that of a reinforced concrete wall, independent of the floor and decking in order to allow free movement. The wall is designed as a cantilever, to withstand the pressure of the water when the pool is full and the earth-pressure

Fig. 8.

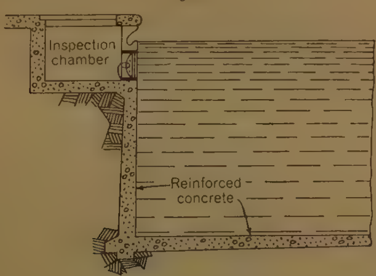


when the pool is empty. Some authorities advocate designing for the latter condition only, but the possible shrinkage of the earth backing makes it advisable to design the reinforcement for both cases.

In a small bath, the wall may be made monolithic with the floor, as shown in *Fig. 9*, but in larger pools the floor is laid over the toe of the wall. The small heel increases the stability of the wall, on account of the earth-pressure when the pool is empty.

If the wall is longer than 100 feet, expansion- (and/or contraction-)

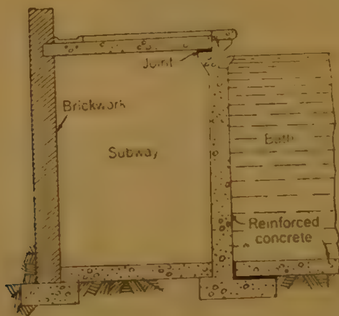
Fig. 9.



joints must be provided to minimize temperature- and shrinkage-stresses. Some designers suggest that joints are necessary every 60 feet of length, but from actual experience this does not appear to be necessary, although some extra longitudinal reinforcement should be allowed. As a useful guide to the thickness of wall required, 1 inch of thickness may be assumed for every foot of water

depth, with a minimum thickness of 4 inches and the provision of the necessary economical reinforcement. The horizontal reinforcement should be at least 30 per cent. of the main reinforcement, in order to reduce shrinkage-cracks. In some designs a subway around

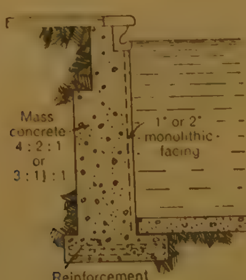
Fig. 10.



the pool (*Fig. 10*) will eliminate external earth-pressure and the decking will support the top of the wall against water-pressure, but even in this case it is advisable to form the wall independently, in order to allow shrinkage to take place.

Mass-Concrete.—Mass-concrete walls are frequently used, and are particularly suitable for open-air pools or constructional work carried out by unskilled labour, not competent to execute reinforced-concrete work. A typical example is shown in *Fig. 11*; the facing

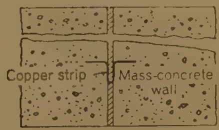
Fig. 11.



was formed in situ, at the same time as the concrete backing, by means of a sliding steel shutter (facings are considered later). Owing to the large volume of concrete there will be a correspondingly large shrinkage, and this is anticipated by alternate-bay construction in lengths of approximately 6 feet. Some form of construction-joint,

such as that shown in *Fig. 12*, is advisable between the sections in order to bond the work together and to safeguard against leakage. Mass-concrete walls have, however, been designed with plain, straight-faced joints. The concrete mix should be kept as dry as is practicable with regard to placing and impermeability, in order to

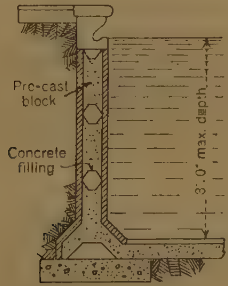
Fig. 12.



reduce the inevitable shrinkage. A suggested mix is 4 : 2 : 1 or 3 : 1½ : 1 for the concrete backing, with a slump of 3 to 4 inches. It is interesting to note that in many baths, where allowance for expansion or contraction has been made, no movement has been perceptible. Tiling or similar facing over the expansion-joints has not moved in any way.

One other form of construction which has recently been introduced is to build the wall of interlocking pre-cast hollow blocks and to place "cages" of reinforcement in the cavities, which are subsequently filled with concrete (*Fig. 13*). It is claimed that this method

Fig. 13.



speeds up construction, and, whilst being cheaper than other methods, is equally effective.

Floor-Slab Design.—The design of the floor-slab is the next consideration. The Author has not considered special cases where external water-pressure is unavoidable, but has assumed that the site has been well drained, in order to obtain a reasonable and uniform bearing from the foundation. If the ground is loose it should be

consolidated by ballasting, but the essential feature is that the bearing power of the foundation must be as uniform as possible. When the foundation has been prepared it should be covered with a 1-inch layer of sand and rolled to a smooth finish; tarred paper should then be laid over this, followed by the reinforced-concrete slab. The concrete should preferably be poured from a shoot and not tipped from a waggon, so that a smooth underside will be obtained for the slab from the undisturbed sub-coat, the slab being thus enabled to move freely under varying conditions of temperature and moisture-content. Owing to the difficulty of assessing the amount of support given by the foundation to the slab, it is usual to employ empirical methods in the actual design of the thickness and reinforcement. For example, a floor-slab 6 inches thick, with top and bottom reinforcement composed of $\frac{1}{4}$-inch bars at 8-inch centres both ways, is equivalent to an excellent road-slab, capable of withstanding heavy traffic and varying temperatures. Although certain of its requirements are very different, the Author considers that a useful analogy can be drawn from the study of a concrete road-slab. The main point of difference is that the floor-slab is subjected to a uniform water-pressure, whilst the road-slab has severe impact-stresses to withstand. The factor of foundation-support is equally indeterminate, and this is the main obstacle to economical and confident design.

If a floor-slab is considered to be totally restrained by the friction between itself and the foundation, a stress of 1,332 lb. per square inch would be induced for a temperature-range of 45° F. (Assuming that the coefficient of expansion of concrete is $7{\cdot}4 \times 10^{-6}$ per °F. and that its modulus of elasticity is 4×10^{6} lb. per square inch, then the temperature-stress $= 4 \times 10^{6} \times 7{\cdot}4 \times 10^{-6} \times 45 = 1{,}332$ lb. per square inch.)

Contraction, not necessarily caused by temperature-changes, will also induce stresses in the slab. If the coefficient of friction between the slab and the foundation is assumed to be 1·25, say, and the weight of concrete is 1/12 lb. per cubic inch, the tension due to friction at 20 feet from the edge of the slab will be

$$1{\cdot}25 \times 20 \times 12 \times \frac{1}{12} = 25 \text{ lb. per square inch}$$

when the pool is empty. When the pool is full, the load will be correspondingly increased, and with a depth of 10 feet of water the induced stress would be

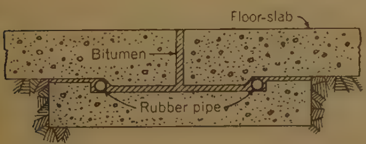
$$1{\cdot}25 \times 20 \times 12\left(\frac{1}{12} + \frac{10 \times 12 \times 62{\cdot}4}{144 \times 12}\right) = 1{,}553 \text{ lb. per square inch}$$

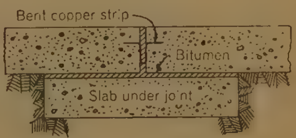
The above stresses are obviously never actually induced, for the tensile strength of concrete is only about 200 or 300 lb. per square inch, but the importance of allowing free movement of the slabs is illustrated.

Factors which may explain the differences between the calculated and the actual results (as known from experience) are that (1) the slab will usually have been constructed at some intermediate temperature between, say, 32° F. and 75° F., and will not therefore expand or contract through the full range of temperature ; (2) the temperature-changes are necessarily gradual and the concrete and foundation may adjust themselves by "flowing" (in a similar way to a metal or glass) ; (3) the effects of shrinkage or moisture will have taken place before the pool is slowly and completely filled ; (4) when the pool is full the water will protect the floor against rapid changes of temperature and moisture-content.

The joints between the slabs must not be spaced too far apart, in order to reduce contraction-stresses, and a maximum size of slab is usually about 30 feet by 20 feet. Practical considerations, such

Figs. 14.





as the shape of the pool, will often decide the layout of the floor-slabs. The actual joint must be water-tight, and yet allow free movement of the slabs. Two types of joints are shown in *Figs. 14.*

To sum up the design of the floor-slabs, the following points should be noted :—

(*a*) A well-drained, solid and uniform foundation is essential ;
(*b*) The coefficient of friction must be reduced to a minimum by means of sand and tarred paper over the foundation ;
(*c*) The slabs must not be too large, the maximum size being, say, 30 feet by 20 feet.

Pool-Lining.—The lining of the pool should be light in colour, in order to demonstrate the purity and clarity of the water, and should present an easily cleaned surface. Tiling or glazed brick linings have been the accepted materials in the past, and have given excellent results, but various cement finishes are now available which are

claimed to fulfil all requirements, such as smoothness and freedom from joints, whilst being cheaper in initial cost. The following linings are typical.

(i) A $4\frac{1}{2}$-inch glazed brick lining may be used, with asphalte between it and the reinforced concrete wall. The glazed brickwork could be replaced by either tiling or tiling on concrete blocks.

(ii) Rendering and tiling can be applied directly to asphalte. The key is obtained by placing patent, dovetailed rubber strips in the warm asphalte, which are removed later.

(iii) The key for the rendering may be obtained by fixing "Hy-Rib" (or similar) reinforcement to the wall.

(iv) Pre-cast units can be supported by "soldiers," to form shuttering for the backing.

(v) By means of a sliding steel shutter, the facing can be poured simultaneously with the grey concrete backing, thus forming a monolithic wall. The floor is constructed in the usual way, with asphalte, covered by 2 inches of facing concrete in situ.

(vi) If cost is the primary consideration, a quite satisfactory finish can be obtained by rendering the mass-concrete wall. Two-coat work is usual, the base coat consisting of 3 : 1 sand and cement up to $\frac{1}{2}$ inch thick, followed by a finishing coat of white cement and silica sand in the same proportions, and from $\frac{1}{8}$ to $\frac{1}{4}$ inch in thickness. The usual precautions, such as "moist curing" and protection from extremes of weather, must be carefully observed. The surface can be highly polished by a mechanical polisher if desired.

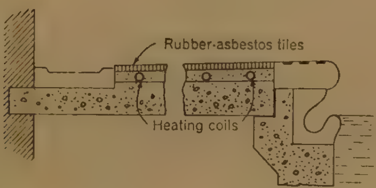
(vii) The cheapest method of finishing is one in which grey concrete is sprayed with a white or coloured cement wash, but this is obviously not very durable, and should not be considered unless grey concrete is the only alternative. The shuttering must be lined with some type of smooth fibre board in order to obtain a good surface, and, as previously mentioned, a higher finish can be obtained by mechanical polishing.

Bath Surrounds.—The surrounds to the pool should be constructed of non-slip material, and should be laid so as to drain away from the pool with a fall of approximately 1 in 60. The coping must be of a distinctive colour, in order clearly to mark the edge of the pool and

so avoid accidents. In a covered swimming bath the surround may be made from 5 to 10 feet wide, but in the open-air pool the width should not be less than 10 feet, and ample open space should be provided for "peak loading" periods.

Various materials have been tried for the pool surround, but few approach the ideals of a non-slip surface combined with impermeability, clean, warm appearance and hard-wearing properties. *Fig. 15* shows a "rubber-asbestos" tile, laid on a concrete screed

Fig. 15.



over heating coils. This method has been successfully employed in a covered bath.

Fig. 16.

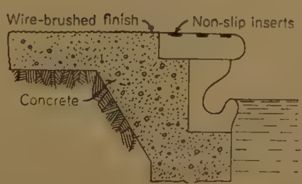
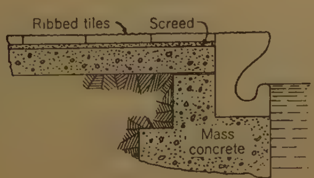


Fig. 16 illustrates how a non-slip finish can be obtained by brushing the plain concrete and dusting over with carborundum.

Fig. 17.

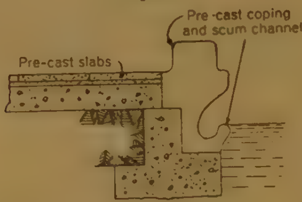


In the method of *Fig. 17* the ribbed tiles are laid with a screed coat over a concrete surround as shown.

Fig. 18 shows a pre-cast unit combining a scum-channel and a coping. The coping prevents the possible contamination of the bath by water from the surround.

The ideal material has yet to be found for the bath surround,

Fig. 18.



although the possible use of rubber-asbestos and asphalte compounds (with asbestos or cork) might be investigated with advantage.

Spectators' Balconies and Terraces.—In the covered swimming bath the spectators will almost certainly be accommodated on balconies along the length of the building, but in the open pool the seating should be arranged in terraces on the amphitheatre system. Dressing-rooms, store-rooms, boiler houses, conveniences, etc., could be arranged under these terraces with advantage, whilst at the same time saving space.

The sight-lines should be designed so that all the spectators have a clear view of the championship area and the high diving-boards. To obtain this the treads of the terraces should be 24 to 30 inches wide and the risers about 18 inches ; a width of 21 inches per person is allowed for the seating. Access-stairs should be provided about every 25 feet of length. These staircases should be 5 feet wide, and in flights of not more than 12 steps of " easy going " (10 inches by 6 inches). The actual seating should be finished in teak, in order to withstand the effects of condensation or weather, and portable cushions should be provided for comfort. In large, open-air pools, the difficulty of seating all the spectators close to the championship area might be overcome by providing floating platforms for the spectators. The platforms could be constructed to form terraces, which could be wheeled into the pool via the shelving beaches and floated into position when required.

Dressing-Hall and Footbaths.—The dressing-halls must be light, well ventilated and warm. Supervision is facilitated if all the

dressing-boxes open into one corridor or hall. The floor must be well-drained and non-slip, for a considerable amount of water is carried back from the pool by the bathers' costumes. The dressing boxes should be constructed of teak, pre-cast terrazzo slabs, or similar durable and easily cleaned material. A special room may be provided for school-children, or for use during rush hours, in conjunction with a locker system. Various locker systems are as follows: a bather changes in a dressing-box and places his clothes in either a steel locker (key provided by the attendant), a paper bag, or on a special hanger, which is in turn given into the care of an attendant.

The remarks as to lighting, non-slip and well-drained floors, durable and impervious materials, etc., apply equally to the room containing the footbaths and showers. In the Author's opinion, the footbath should be replaced by an efficient range of shower cubicles, in order to discourage the schoolboy from "soaking" in the hot-water baths. The shower cubicles should be fitted with a mixing valve to supply hot, tepid, or cold water, a soap receptacle and shallow footbath (say 4 inches deep). Footsprays, operated by the pressure of the bather's heel, have been installed in recent swimming baths and have proved very popular.

In open-air pools, showers are almost as essential as in indoor baths, and a feature could be made of an open-air shower, which would at the same time act as an aerator for a small proportion of the filtered water.

The footbaths and showers should be made compulsory, in order to maintain the water of the swimming-bath at a high standard of purity and thus to protect bathers from infection. The dressing-boxes should be arranged with two entrances, one from a corridor reserved for bathers entering the boxes (with boots) and the other reserved for bathers about to enter the water. Another system would be to have an undressing hall, and a dressing hall, in conjunction with the locker system (which is now almost universal). After leaving the dressing box the bather must proceed to the footbaths and showers, and thence through an unavoidable footbath and shower to the pool. The shower could be operated by the weight of the bather on the floor.

Details.—There are many details in the design of a swimming pool which demand more than ordinary attention, in order to lessen the risk of accidents and infection. The ladders leading from the water, for example, must be designed so that a bather may not trap his foot or bruise his leg if he falls back into the water.

The scum trough also, in addition to its primary function, may serve as a step for leaving the pool, and the above remarks must, therefore, be considered in its design.

Filtration, Heating, Ventilation and Lighting.

Whilst this Paper is not primarily written to deal with the above subject, it is felt that some brief comments on modern features should be included.

Filtration.—The question of an inward "lateral-flow" filter is being investigated at the present time by a well-known engineering firm in Lancashire, and, whilst this principle is only in the experimental state, it is felt that the advantages to be obtained from the increased area of filtering media will merit its adoption in the near future. The advantages of reversing the direction of flow of the circulated water also seem worthy of consideration. In this method the warm, filtered water is admitted at the lowest point (or points) of the pool and drawn off at surface level. The deeper parts of the pool are warmed and heat-losses are reduced, and the inevitable surface scum, dust, etc., is withdrawn as it collects. Under the existing system the deep water is cooler than the shallow water, and surface filth must be removed by skimming. Surface filth is the most dangerous, for it can so easily be swallowed by the bather. This scum is particularly noticeable where "underwater lighting" is in use. Another advantage of the reversed system where chlorination is employed is that as the warm water cannot hold chlorine so well as the cold water, the chlorine tends to be evenly distributed throughout the full depth of the pool. Under existing conditions there is excess, and therefore unpleasant, chlorine on the surface, and a deficit in the deeper water.

The "Katadyn" process for sterilizing the water has been recently introduced, but it has not yet been sufficiently proved for universal adoption. The process depends upon the introduction of silver ions by means of an electrolytic apparatus, and, unlike chlorination, an overdose has no unpleasant effects. Other methods of sterilizing are: (1) with chlorine, the usual present-day method; (2) with chloramine, a mixture of chlorine gas with ammonia; (3) with ultra-violet rays, a method which is still in its infancy; and (4) by the agency of ozone produced electrically.

In the future facilities will no doubt be provided for the sterilizing of bathers' costumes before they enter the water, but many conventions will have to be broken down before this is feasible.

Heating.—Whilst the water in all indoor baths is heated, the heating of open-air pools is comparatively rare. The usual season for an open-air pool is only 3 months, and this could be considerably lengthened by means of artificial heating ; any methods which make this more economical and practicable are worthy of consideration. If the water is heated, the heat must be conserved by insulating the floor and sides of the tank by means of cork slabs (protected with bitumen or asphalt) and by special attention to the provision of a well-drained, porous subsoil. Even when the tank is insulated, however, the main heat-losses will be from the surface of the water during the night, when the air-temperature will be at its lowest.

To overcome this, the Author suggests that the pool could be covered by means of tarpaulins suspended on wires about 12 inches above the water-level. Alternatively, the water could be protected by floating mats composed of cork covered with rubber, but this method would be more costly and more storage space would be needed during the daytime. The bath must also be protected from cooling winds, not only for the above reasons, but also in order to protect the bather, and this can be accomplished by means of terraces and glass screens.

By the above methods the bath would become a veritable "thermal-storage tank," which, if electrical heating is employed, could be heated during the night with the cheapest electric current available.

Ventilation.—In the covered winter swimming-bath little attention has been paid in the past to air-conditioning and the provision of a warm, fresh atmosphere, with the result that winter swimming has never become popular with the general public. The use of double glazing (with a heated air-space between), together with scientific air-conditioning, can do much to overcome the existing conditions.

Lighting.—A swimming-pool, as befits a place of healthy recreation, must be well lighted, not only at night, but also during the daytime. A window-area of at least half that of the pool and surrounds is recommended, and, whilst this should be easily exceeded in the case of a glazed roof, a building without roof-glazing must be provided with tall windows from floor to ceiling. These windows could be made to open in warm weather to provide open-air conditions and access to sunbathing beaches and lawns. It appears to be only a matter of time before glass transparent to ultra-violet rays will be

used universally at an economical price, for its health-giving properties are almost too well known to need mention.

The artificial lighting of the covered pool is often done by means of pendants, which, although cheap, are not to be recommended on account of inaccessibility and the danger of breakages, and hence the risk of injury to bathers from broken glass. Lowering gear is also necessary with pendant lighting. The lighting should preferably be indirect from the ceiling and wall panels, thus protecting the fittings from dampness and also preventing glare and dazzling reflections. As a general rule all fittings should be out of reach of the bathers for reasons of safety.

In the open-air pool the lighting should be from tall standards, placed well away from the edge of the water, and, as in the covered pool, all dazzle and glare should be minimized. In both types of pools, underwater lighting is becoming popular, for it adds greatly to the attractiveness of the pool, especially at night, and avoids surface reflections.

There is great scope for the floodlighting of buildings (usually white or light in colour), and for the coloured lighting of aeration-fountains and similar features.

Wave-Making Machinery.—Although artificial waves are quite common in Continental swimming-pools, they have only recently been introduced into England and have only achieved a doubtful popularity. The Empire Swimming-Pool at Wembley is fitted with a wave-making plant consisting of four hydraulically-operated plungers moving in chambers with an up-and-down motion. The speed and stroke of the plungers is so arranged as to have a cumulative effect on the waves, which finally break upon a beach at the opposite end of the pool. At the Luna Park Swimming-Bath in Berlin, artificial waves are produced by means of a large "hinged shutter," which is slowly withdrawn and then quickly jerked forward in order to produce the wave motion. Here again the speed and stroke will affect the size of the waves. Incidentally, the "surf" is here produced by an enlargement of the pool from a rectangle to a circular basin at the opposite end. It is stated that in the case of an open-air pool at Zurich (approximately 120 feet by 60 feet) a 40-HP. motor was required to produce waves 3 feet high.

Owing to the height of the waves the surrounds must be raised considerably above the normal water-level, and this is preferably done by terracing from above wave-level down to the usual bath surround. The carcase of the bath must also be designed to withstand the impact caused by the waves.

Continental Swimming-Pools.

Many of the Continental swimming-pools embody features which might usefully be studied and adapted to English conditions and requirements.

Germany.—The open-air swimming-pool at Munich is a typical example, which embodies the following interesting features :—

(*a*) Large, open, sunbathing-lawns, 3 or 4 acres in extent, with facilities for exercise in the form of horizontal bars, jumping-pits, etc. ;

(*b*) A football-pitch and running-track arranged so as to form part of the scheme ;

(*c*) The main swimming-bath, divided into two separate pools, one for diving and one for swimming ;

(*d*) Two separate children's baths, each with sloping beaches ;

(*e*) An open-air restaurant, which is railed off, and has accommodation for about 200 people ;

(*f*) A shelf (9 inches wide), about 4 feet below water-level, alongside the walls, to enable bathers to rest without leaving the water ;

(*g*) Numerous sun-bathing "racks," which allow the bather to dry out in the sunshine without coming into contact with the ground ;

(*h*) Notice-boards, with exercises enumerated and described.

The "locker system," as adapted at Munich, is as follows : the bather enters any vacant dressing-box and hangs his clothes on a wire hanger, which is then handed through a small door at the rear of the box to an attendant in the store room.

Mention has already been made of the wave baths at Luna Park, Berlin, and Germany has many other modern swimming-pools, such as the open-air bath at Frankfurt and the indoor bath at the Sportforum, Berlin.

France.—The municipal swimming establishment at Bordeaux comprises a covered winter swimming-pool and a summer open-air pool. A large gymnasium is provided in an adjoining building, and sunbathing terraces lead down to a large expanse of sports grounds. The most interesting feature of this scheme is probably the sliding doors at one end of the winter pool, which combine the two baths

during the fine weather. The doors are 12 metres in height, about 15 metres in total width, and are duplicated in order to form an insulating, heated air-space in the colder weather. The sliding gear is controlled by hand and the doors are completely hidden in recesses when opened. The roof is carried by an N-type girder, and the suspended ceiling is composed of porous concrete blocks. The ceiling is so designed that any condensation will be absorbed and carried up by capillary attraction until it is dried out by the current of warm, dry air which is made to flow between roof and ceiling. The concrete carcase of each swimming pool is mainly supported on piers and arches, is designed to act independently of surrounding structures, and is insulated against heat-losses.

Denmark.—A quite small, yet ideal, bathing pool is attached to the College of Physical Culture at Ollerup. This bathing pool, constructed on a hillside, is comprised of two pools. The lower is the open-air swimming pool and is connected to the indoor bath by means of a channel about 9 feet wide. Owing to the contours of the site, the indoor bath is almost entirely below ground-level, and spectators can watch the bathers through deck-lights, as though from a roof-garden. Here again a gymnasium is at hand and also a covered, superelevated running-track, to protect the pool on the exposed sides.

Switzerland.—The Swiss lakeside bathing places are worthy of mention. The large terraces, open-air cafés and sunbathing lawns are typical features, and provision is made for boating and outdoor games. It is doubtful, however, whether English lakes would be so popular, even if adapted, owing to the different climatic conditions.

Holland.—An ultra-modern swimming-bath has been built at Arnhem, in Holland, in which a totally sliding roof is the principal feature, but the Author has unfortunately been unable to obtain particulars of its construction.

Belgium.—The idea of a glass-sided swimming-tank has already been exploited at Ostend, where diners in a café may watch the swimmers through a large glass panel.

Japan.—A swimming-tank, constructed entirely of glass, has been built in Japan, but details are unobtainable at the moment.

A general feature of Continental swimming-pools is that more thought is given to the bather than the spectator, whilst in many English pools the bather is allowed little space for exercise, in order that more spectators may be accommodated.

Acknowledgements.

In conclusion, the Author would like to express his thanks to all those persons who have so willingly supplied information, and in particular to Mr. E. L. Leeming, M.Sc., M. Inst. C.E., for his many suggestions, and also to Mr. G. L. Goulden, Assoc. M. Inst. C.E., for the information on the open-air pool at Munich.

The Paper is accompanied by 22 illustrations, from some of which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

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"The Reduction of Carrying Capacity of Pipes with Age."

By CYRIL FRANK COLEBROOK, Ph.D., B.Sc. (Eng.), Stud. Inst. C.E., and Assistant Professor CEDRIC MASEY WHITE, Ph.D.

(*Ordered by the Council to be published with written discussion.*) [1]

TABLE OF CONTENTS.

INTRODUCTION.

The Paper outlines a theory based on the von Kármán–Prandtl resistance law, together with the hypothesis that the hydraulic deterioration of pipes is entirely due to increase in surface-roughness, and that the size of the roughness-irregularities grows in direct proportion with time. This assumption leads to a certain difficulty with regard to conditions when the age of the pipe is zero, but this is overcome by including a term involving the initial Chezy coefficient C of the pipe. A formula is developed giving the relation between the age of a pipe and its carrying capacity (p. 116), and a tabular statement (pp. 112–13) gives the rate of growth of irregularities under various conditions. A convenient approximate formula is set out on p. 116.

THEORY OF THE EFFECT OF ROUGHNESS IN PIPES.

IT was, until recently, impossible to find a rational basis for predicting the loss of carrying capacity of water-mains, as the mechanism by which surface-roughness caused fluid friction was not sufficiently understood. However, in 1933 Prandtl,[2] using his own and von

[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—ACTING SEC. INST. C.E.

[2] L. Prandtl, "Neuere Ergebnisse der Turbulenzforschung." *Zeit. Ver. deu. Ing.*, vol. 77 (1933), p. 105.

Kármán's theoretical contributions in conjunction with the experimental results of Nikuradse, was able to explain quantitatively the parts played by surface-roughness and by viscosity in resisting fluid motion. This contribution has received much notice [1] and it is only necessary here to give the barest outline of the relevant parts of the theory.

There are several theoretical lines of approach, but all tacitly rely ultimately upon the experimentally-observed fact that within the main body of fluid in turbulent motion the velocity-distribution does comply fairly well with the relation

$$\frac{du}{dy} = \sqrt{\frac{\tau}{\rho}} \cdot \frac{2{\cdot}5}{y} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where u denotes velocity at a distance y from the wall of the pipe, τ denotes shear stress at the wall, and ρ denotes density of the fluid, and in which the so-called "universal constant," 2·5, is derived from experiment, and may perhaps be regarded as implying that the effective size of an eddy or disturbance at any point is 1/2·5 of the distance of this point from the wall which is generating the disturbance. But at the pipe-wall this relation fails, because it does not recognize any mechanism for resisting the motion there. It ignores the turbulent mixing found among the irregularities of a rough wall, and it ignores the molecular or viscous mixing found beside a smooth wall, so with $y = 0$ the simple relation (1) erroneously gives $\frac{du}{dy} = \infty$. Now, one assumption on which it is based is that the size of the turbulence-disturbances increases up to the centre of the pipe in direct proportion to the distance away from the wall; accordingly, it is always possible to select some plane parallel to the wall, but displaced inwards, where the disturbances in the theory are as great as the real ones are at the wall. A minute displacement is sufficient—a matter of less than 0·001 inch even in a fairly rough concrete pipe. There is, therefore, no practical need to complicate the theory by adding terms to represent mixing at the wall, since it is merely necessary to regard the effective hydraulic wall as being shifted inwards by an appropriate small amount y_1, and the infinite-velocity-gradient difficulty then disappears. Alternatively, the mathematical origin for y can be taken a similar small distance outside the pipe.

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[1] H. Rouse, "Modern Conceptions of the Mechanics of Fluid Turbulence." Proc. Am. Soc. C.E., vol. 62 (1936), p. 21.

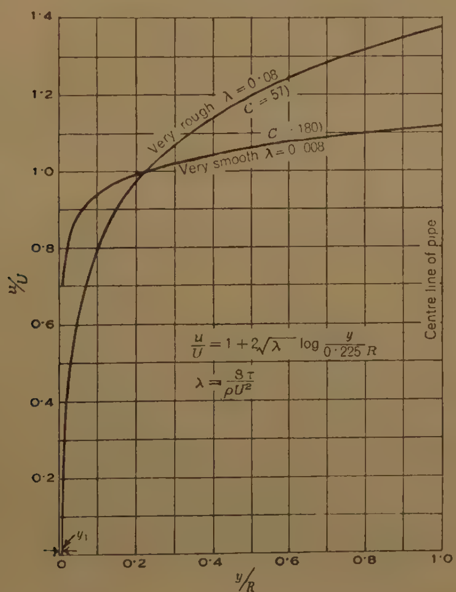
A. Fage, "Aerodynamical Research and Hydraulic Practice." Proc. Inst. Mech.E., vol. 130 (1935), p. 3.

On integrating equation (1) between limits and assuming that the velocity is zero at $y = y_1$, the velocity-distribution is found to be

$$u = 5{\cdot}76\sqrt{\frac{\tau}{\rho}}\log\frac{y}{y_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1\text{A})^*$$

Fig. 1 shows velocity-distribution curves calculated from this relation, the two examples relating to extreme conditions, one for very low and the other for very high friction.

Fig. 1.



VARIATION OF VELOCITY ACROSS THE PIPE.

(Based on equation (1A) after eliminating y_1 by writing $u = U$ when $y = 0{\cdot}225R$ as given by equation (1B).)

Now, by definition, the mean velocity U in a pipe of radius R is given by

$$U = \int_0^{R-y_1} 2\pi u r dr / \pi R^2$$

---

* The logs have been converted to base 10, the factor 5·76 being 2·5 × 2·3026.

which, on substituting the right-hand side of equation (1A) for u, becomes

$$U = 2{\cdot}5\sqrt{\frac{\tau}{\rho}}\int_0^{R-y_1}\frac{2r}{R^2}\log_e\frac{R-r}{y_1}\,.\,dr,$$

which after integration becomes

$$U = 2{\cdot}5\sqrt{\frac{\tau}{\rho}}\left(\log_e\frac{R}{y_1} - 1{\cdot}5 + 2\frac{y_1}{R} - \frac{y_1^2}{2R^2}\right).$$

This, on converting to base 10, and dropping negligible terms in y_1/R, simplifies to

$$U = 5{\cdot}76\sqrt{\frac{\tau}{\rho}}\log\frac{0{\cdot}225R}{y_1}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (1\text{B})$$

from which, by comparison with (1A), it is evident that the local velocity is numerically equal to the mean velocity when $y = 0{\cdot}225R$. The value for $\log y_1$ from (1B) may be substituted back in (1A) which then takes the form given in *Fig. 1*.

Equation (1B) may be written somewhat more conveniently so far as arithmetic is concerned by breaking up the 5·76 into $\sqrt{8}$ and 2 approximately, thus

$$\sqrt{\frac{\rho U^2}{8\tau}} = 2\log\frac{0{\cdot}225R}{y_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Since in unaccelerated flow the applied pressure-gradient is balanced by the frictional resistance, that is, by the shear stress τ acting over the surface of the pipe, it follows that $\tau = \frac{\rho ghd}{4l}$, where h denotes the loss of head in a length l; hence the group $\frac{8\tau}{\rho U^2}$ is identical with the coefficient λ in the well-known pipe-resistance formula $h = \frac{\lambda\rho U^2}{2gd}$, and $\sqrt{\frac{\rho U^2}{8\tau}}$ only differs from the Chezy coefficient in that the latter omits $\sqrt{8g}$ which is contained in the former. Thus in Chezy's formula, which is $U = C\sqrt{mi}$, C is given by

$$C = \sqrt{8g}\sqrt{\frac{\rho U^2}{8\tau}}.$$

It is therefore evident that equation (2) completely determines the resistance-law, provided that y_1 can be assessed. Since the shift y_1 is only concerned with conditions at the wall, the factors on which it can depend are restricted to (a) the roughness of the wall, (b) the intensity of the shear stress, and (c) the viscosity of the fluid; and there are theoretical grounds for expecting (c) to be unimportant

when (*a*) and (*b*) are comparatively large. Under these circumstances, dimensional reasoning shows that y_1 must be proportional to k, the size of the roughness-protuberances, provided that their geometrical shape is unchanged.

Nikuradse's experiments with a series of pipes artificially roughened internally by a layer of sand fixed to the walls confirm this deduction, and show that, in the case of the particular sand and spacing used by him,

$$y_1 = \frac{k}{33}, \quad \dots \dots \dots \quad (3)$$

where k denotes the measured diameter of the sand-grains. Inserting (3) in equation (2), the general resistance law becomes, as a particular case,

$$\sqrt{\frac{\rho U^2}{8\tau}} = 2 \log \frac{3{\cdot}7d}{k} \quad \dots \dots \dots \quad (4)$$

or

$$C = 2\sqrt{8g} \log \frac{3{\cdot}7d}{k}$$

An experiment by Schlichting [1] with Hamburg sand, and one by the Authors [2] with Aylesford sand, gave $y_1 = \frac{k}{20}$ and $y_1 = \frac{k}{25}$ respectively, the differences in all probability being due to variations in the uniformity of the sand-layers rather than to variation in sand-grain shape. However, these differences only have significance when the actual physical size of the roughness has been measured. For the present treatment it is convenient to use $y_1 = k/33$, as this facilitates comparison with Nikuradse's sands. In any case, both theory and experiment show that y_1 is independent of the size of the pipe, so a single measurement of the resistance of one particular pipe can be used to predict the resistance of pipes of other sizes provided that all have identical surfaces.

Practical Considerations.

After water-mains have been in service for some time, their hydraulic resistance usually increases owing to growths or deposits upon the internal surfaces, and unless the head is increased the flow falls off until eventually cleaning or replacement is necessary. The time–deterioration relation appears to depend mainly upon the character of the water, the size of the pipe, the material and lining

[1] H. Schlichting, "Ein neues Verfahren zur Messung des Strömungswiderstandes von rauhen Wänden." *Werft Reederei Hafen*, vol. 17 (1936), p. 99.

[2] "Experiments with Fluid Friction in Roughened Pipes." Proc. Roy. Soc. (A), vol. 161 (1937), p. 367.

used, and the velocity. With so many variables involved, no single set of records can be exhaustive, but, as the following treatment shows, simplifying assumptions can be made which enable records taken over short periods of time in one particular pipe to be used to predict future changes not only in this pipe but in all others of similar type in the same district.

Application of Theory.

Loss of carrying capacity may be due to an increase in roughness or to an actual reduction in cross-sectional area, or to a combination of both, but by comparison the effect of loss of area is so unimportant that it can almost be ignored, except perhaps in extreme cases. This rather surprising result follows from equation (1B), which on substituting $\rho gdi/4$ for τ, Q_0/a for U, and $k_0/33$ for y_1, becomes

$$Q_0 = a\sqrt{8gdi}\log\frac{3{\cdot}7d}{k_0}.$$

Supposing that with age the diameter d decreases to $d - s$, and the roughness k_0 grows to k, then the flow will be reduced to

$$Q = a\sqrt{8gdi}\left(1 - \frac{s}{d}\right)^{\frac{5}{2}}\left\{\log\frac{3{\cdot}7d}{k_0} + \log\frac{k_0}{k} + \log\left(1 - \frac{s}{d}\right)\right\} \quad . \quad (5)$$

where Q denotes discharge, a denotes initial area of pipe, d denotes initial diameter of pipe, i denotes hydraulic gradient, k_0 denotes initial roughness-size, k denotes roughness-size after period of service, and s denotes loss in diameter. Thence it is seen that if the diameter remains constant and only the roughness changes, then, since $C = 2\sqrt{8g}\log 3{\cdot}7d/k$,

$$C_0 - C = 32\log\frac{k}{k_0}\ (\text{feet}^{\frac{1}{2}}\ \text{seconds}^{-1}),$$

which for a tenfold increase in roughness, say, from 0·01 inch to 0·1 inch, gives a reduction of 32 in C, representing between 20 and 30 per cent. loss in the carrying capacity of the pipe, caused by a growth in irregularity somewhat less than 0·1 inch. Had this same growth of 0·1 inch been uniform, reducing the diameter from, say, 20 inches to 19·8 inches without changing the roughness, then C would have been decreased approximately by 0·1, the diameter by 1 per cent. and the area by 2 per cent., causing a total of only 2·6 per cent. loss in carrying capacity.

These typical values show that the deterioration can be regarded without serious error as due entirely to increase in surface-roughness, an assumption which greatly simplifies the treatment.

Growth of Irregularities with Age of Pipe.

After the first few months of service it is reasonable to suppose that the rate of growth becomes constant and that the size k of the irregularities increases in direct proportion with time t. This assumption is on the face of it the simplest that can be made. With regard to its validity little can be said save that experimental records show unmistakably that deterioration is certainly some function of time, although, except for the Thirlmere data referred to later, they are not precise enough to give much guide as to the form of the function. Perhaps the assumption of the linear relation can best be regarded as a first approximation to the real law, whatever this may be. Admittedly, future experiments may disclose other more complicated relations, but for the present the best assumption is that the size of the irregularities varies directly with the age of the pipe.

If this is reckoned from a suitable date, not necessarily that at which the pipe is first used, then the relation takes the simple form $k=\alpha t$, in which the growth-rate α depends primarily upon the character both of the water and of the pipe-lining. Any imperfections of the theory as well as neglected factors must also be included as possible sources of numerical changes in α, but these appear small compared with the larger changes from district to district. On the whole α is best regarded as an experimental coefficient to be determined from analysis of records of deterioration in each district.

It remains only to discuss how y_1 depends upon k. As mentioned earlier, Nikuradse's experiments with sand-roughened pipes prove that for them y_1 is proportional to k, provided that the velocity is high enough for the resistance to be proportional to the square of the velocity; this was so in those experiments when

$$\frac{\sqrt{\tau/\rho}\,k}{\nu} > 60 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6\text{A})$$

which may be written in another form as

$$R > 90 \, . \, p \, . \, 10^{p} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6\text{B})$$

where p denotes $\dfrac{C}{2\sqrt{8g}}$, C denotes the Chezy coefficient, R denotes the Reynolds number (vd/ν, in which v denotes the velocity, d the diameter, and ν the kinematic viscosity).

It must be borne in mind that the numerical values in these expressions are based on experiments with roughnesses of very regular and uniform type, and with grains closely adjacent to one another, whereas the tubercles which form in old pipes are often far apart, leaving relatively smooth stretches in between. However, experiments by the Authors have shown that the limit represented

by equation (6) is also approximately valid even when the sand grains are spaced so far apart that 95 per cent. of the pipe surface remains smooth; so the same limit is probably true enough for age-roughened pipes.

This conclusion is well supported by the published records for old mains, typical selections of which are represented in *Fig. 2*, where the resistance-coefficient λ is shown plotted logarithmically on a base of Reynolds number. The curves relate to old uncoated cast-iron, asphalted cast-iron, and wrought-iron pipes, and the dotted curve sloping downwards across the diagram represents the limit of equation (6). The majority of the curves to the right and above the dotted curve are horizontal lines, showing that the resistance-coefficient is sensibly constant for each pipe and that the resistance in this region is of the required square-law character. On the other hand, as explained in an earlier Paper,[1] there is a wide transition-range lying below and to the left of the dotted curve; and most new pipes fall in this range, in which λ is influenced not only by surface-roughness but also to some extent by viscosity, speed, and pipe-size. However, the present discussion mainly concerns old pipes, and *Fig. 2* does confirm that these obey the square law.

There would thus be a *prima facie* case for expecting y_1 to be proportional to k and so to t, if only the geometrical form of the roughness did not change with time. Unfortunately, geometrical change does take place with that type of roughness formed by isolated nodules, since the ratio of nodule-size to pitch becomes greater with increasing size of nodule.

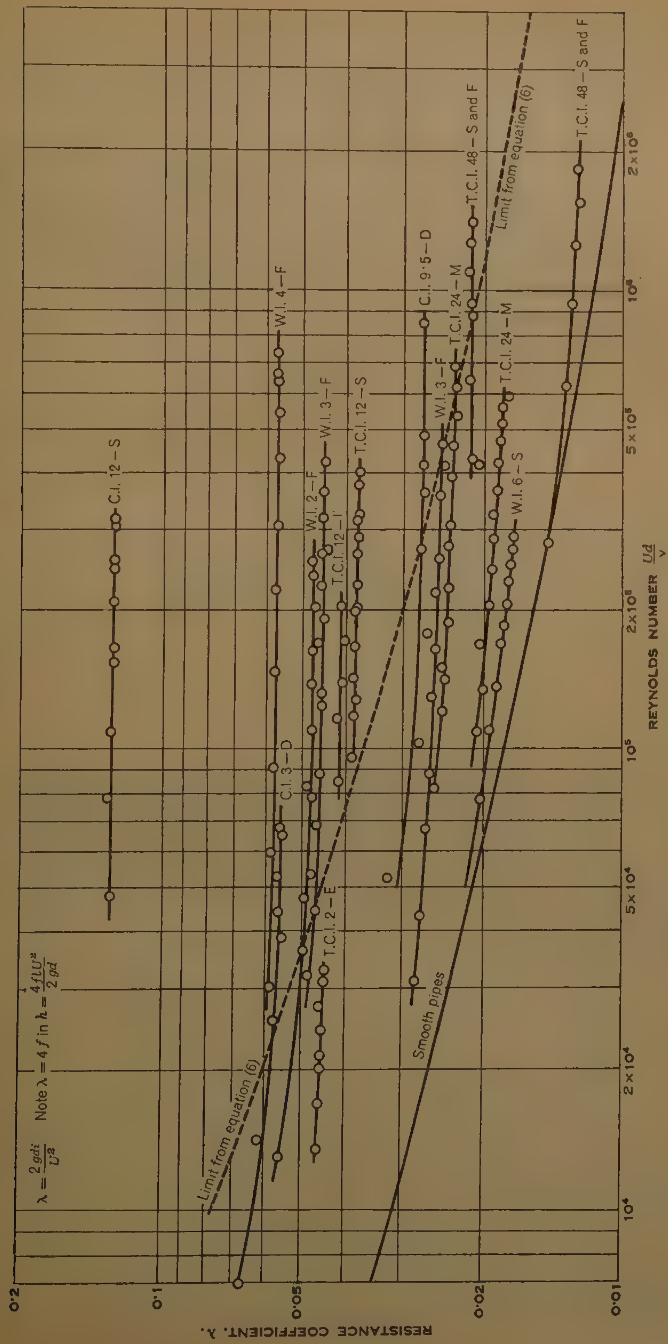
Schlichting's experiments with regularly-spaced rivet-heads and analogous protuberances of size k show that the ratio y_1/k at first increases as the pitch is reduced, then reaches a maximum and subsequently diminishes until the protuberances touch. This is with regular spacing. With chance spacing, when some are very near together and others far apart, it would be expected that the two effects would partially neutralize, leaving y_1/k more nearly constant than Schlichting observed. Hence, until experimental evidence provides more precise relations it is justifiable to use the simple ones, namely, that y_1 is proportional to k, and k in its turn is proportional to t.

Probably the most reliable of the ageing experiments carried out in Great Britain are those by Barnes [2] on the Thirlmere Aqueduct. The records taken at intervals of a few months extend over 14 years,

[1] Footnote (²), p. 103.

[2] A. A. Barnes, "Discharge of Large Cast-Iron Pipe-Lines in Relation to their Age." Minutes of Proceedings Inst. C.E., vol. ccviii (1918–19, Part II) p. 1.

Fig. 2.

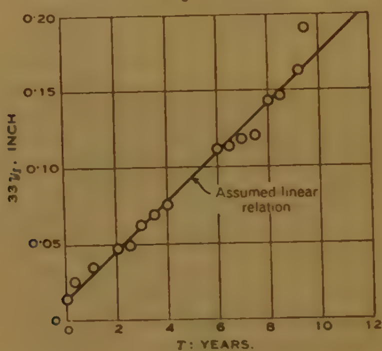


RESISTANCE-COEFFICIENT AND REYNOLDS NUMBER FOR OLD MAINS.

and show the gradual increase in resistance of two tarred cast-iron pipes lying side by side and supplied with the same water. The pipes differed in diameter, one being 44 inches and the other 40 inches, and differed in age by 10 years. The test-points for the older 40-inch pipe are not so complete for its early life, and they also show greater experimental scatter, so perhaps it is best to confine attention to the 44-inch pipe, although in any case both behave in much the same way.

Fig. 3 has been prepared from the data for the Thirlmere 44-inch pipe to show the manner in which y_1 actually does increase with age. The test-points are seen to exhibit an unmistakable tendency towards a linear dependence upon time. Some experimental scatter

Fig. 3.



is unavoidable, since, apart from possible seasonal variations, a small error in measurement, perhaps a matter of 1 per cent. only, could quite well cause an error of 10 per cent. in the apparent value of y_1. In view of this, *Fig. 3* must be regarded as lending remarkable support to the theory and justifying the assumption that y_1 varies directly with t.

With $y_1 = k/33$, equation (1B), on substituting $U = Q/a$ and $\tau = pgdi/4$, becomes

$$Q = a\sqrt{8gdi}\log\frac{3{\cdot}7d}{k} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

which with $k = \alpha t$ becomes

$$Q_t = a\sqrt{8gdi}\left(\log\frac{d}{t} + A\right) \quad . \quad . \quad . \quad . \quad . \quad (8)$$

where $A = \log 3{\cdot}7/\alpha$ or

$$Q_1 - Q_2 = a\sqrt{8gdi}\log\frac{t_2}{t_1} \quad . \quad . \quad . \quad . \quad . \quad (9)$$

where Q_t denotes the discharge at time t, and A is an "ageing constant" which should be applicable throughout any one district.

In these relations the question of the time-origin requires discussion. It will be noticed that the formulas fail as t approaches zero, the reason being that the theory ignores the resisting mechanism of the new pipe and assumes that $y_1 = 0$ when $t = 0$. Even if the new pipe is perfectly smooth y_1 will be finite owing to the action of viscosity, so the formulas require modification to take the original state into account. This may be done by selecting a suitable date from which to reckon the age of the pipe, some weeks or months being added to its true age, so that in the theory y_1 may have had time to grow to the size required to represent the resistance of the new pipe. The necessary shift t_0 of the time-origin is given by

$$t_0 = \frac{k_0}{\alpha}, \quad \text{but } C_0 = 2\sqrt{8g}\log\frac{3{\cdot}7d}{k_0}, \quad \text{so}$$

$$t_0 = \frac{3{\cdot}7d}{\alpha 10^{p_0}}, \qquad \ldots \ldots \quad (10)$$

where $p_0 = \dfrac{C_0}{2\sqrt{8g}}$, which completes the theory by providing the appropriate time-scale in terms of the Chezy coefficient C_0 of the new pipe and α, the rate of growth of roughness. Equation (9) can now be rewritten as

$$Q - Q_0 = a\sqrt{8gdi}\log\frac{t_0}{t}$$

or

$$\frac{Q}{Q_0} = 1 - \frac{1}{p_0}\log\frac{t}{t_0} \quad \ldots \ldots \quad (9\text{A})$$

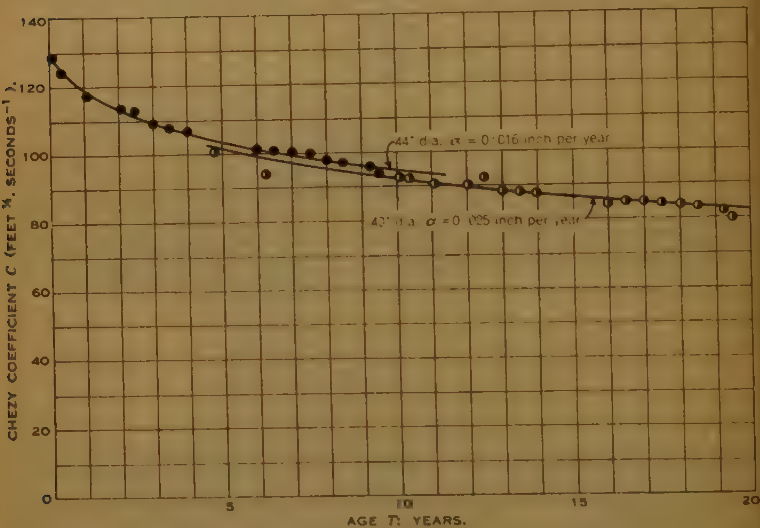
Fig. 4 (p. 110) includes all the Thirlmere test-points, and these are seen to be in complete agreement with the theoretical curves; in fact, in the light of *Fig. 3* this agreement was inevitable. The position is not so clear, however, with regard to most of the other published data, as these are very irregular, probably being influenced by seasonal temperature and chemical variations superimposed upon the already large experimental errors to be expected in such work. In many cases there are no records of the behaviour of the pipes when new, nor are the ages given precisely. However, despite these uncertainties there does appear to be a significant correlation between the theory and observation, sufficient indeed to indicate the probable numerical range of the coefficients.

A lengthy report issued by a Committee of the New England Water Works Association in 1935 contains a record of American experience of ageing effects, though unfortunately based on the Williams-Hazen formula,[1] which is unsuitable since its coefficient is not independent

[1] $U = Wm^{0{\cdot}63}i^{0{\cdot}54}\,0{\cdot}001^{-0{\cdot}04}$. Journal New England Waterworks Association, vol. 49 (1935), p. 239.

of the speed of flow. The speeds were not given, so to convert to Chezy coefficients it has been necessary to assume that the velocity was 4 feet per second. A typical selection of data so converted is presented in *Fig. 5*, which shows the observed variation of C as a function of age in years reckoned from the time the pipes first went into service. The heavy curves are drawn in accordance with the theory, with α and t_0 selected to give the best agreement. In view of the methods by which most of the records were obtained it is seen that the fit is as good as could be expected.

Fig. 4.



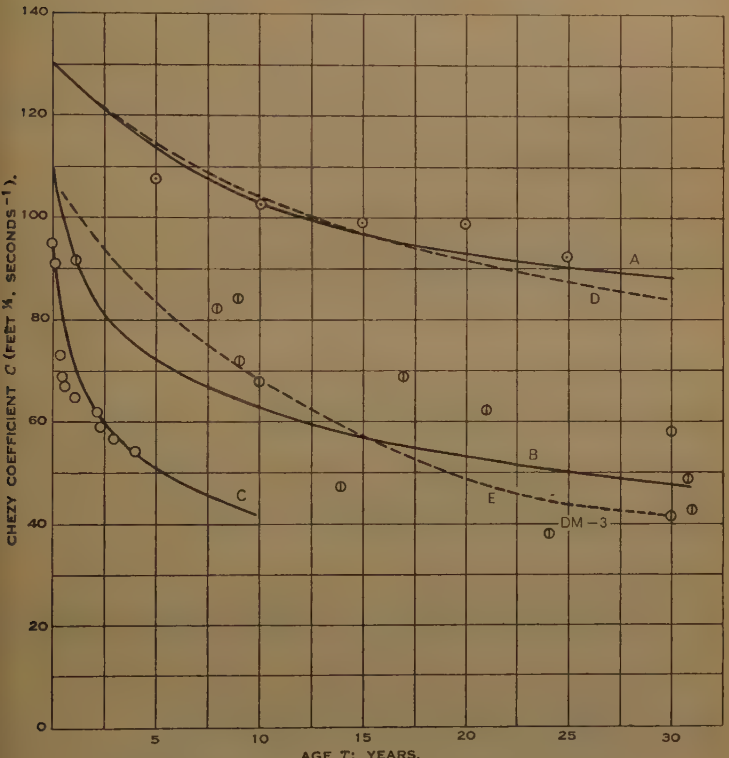
The theory requires that the quantity should fall off in direct proportion to the logarithm of the age when the latter is reckoned from the shifted origin as explained earlier, and a more drastic test is to plot the curves on a base of log (time); the points should then lie on straight lines as indicated in *Figs. 6* and *7* (p. 114). Here, although the agreement is not so apparent, it is seen that the points do on the whole exhibit the required linear relation, or at least sufficiently so to justify the theory in the absence of a better one.

The Authors have accordingly analysed the published records, and they find the growth-rates shown in Table I (pp. 112–13). The values range from 0·0025 inch per year to 0·08 inch per year, with a mid-value of 0·015 inch per year, half the districts having growths lying between 0·006 and 0·025 inch per year. The American committee arbitrarily divided the pipes into two groups, as those less than 16 inches in diameter deteriorated far more rapidly. This distinction

seems unnecessary, as part of the difference is obviously due to different waters, and the remainder no doubt to different velocities, temperature-ranges, and errors inherent in the Williams-Hazen formula.

The growth-rates decrease with increasing pH value, but the

Fig. 5.



TYPICAL AMERICAN DATA IN COMPARISON WITH THEORY.

Curves A, B and C : theoretical.

,, D and E : empirical, as found by N.E.W.W.A. Committee.

ⓘ Data on which curve E was based : cast-iron pipe 6·9 inches average diameter. Each point relates to a different pipe.

⊙ Rochester data : riveted steel pipe 38 inches diameter.

○ St. Louis, Mo., data : 6-inch pipe after cleaning.

scatter in *Fig. 8* (p. 115) shows that other equally important factors are present, and until these are understood any curve purporting to represent the influence of pH can be little better than a guess ; still, for what it is worth, it may be mentioned that the line

$$2 \log \alpha = 3{\cdot}8 - p\text{H}$$

passes well among the points in *Fig. 8*. In one district the water,

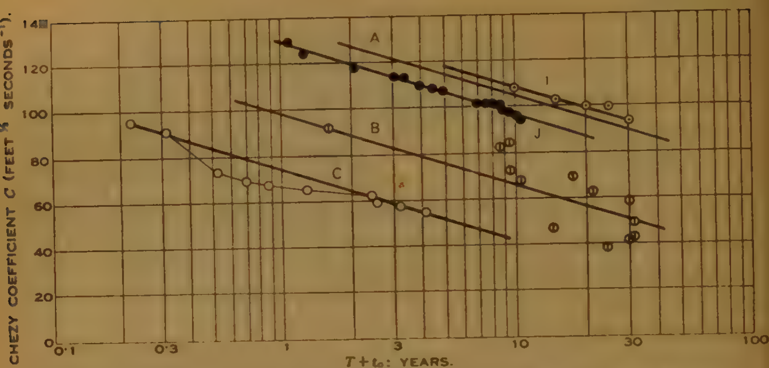
TABLE I.—COLLECTED DATA RELATING TO THE DETERIORATION OF PIPES. AND (8) (P. 108)

Town reference.	District, source of water, etc.
S.L. 1	Great Lakes, raw river water . . . .
S.M. 1	Mississippi Basin, Ohio river. Coagulated, lime, ferrous sulphate, filtered . . . .
S.M. 2	Mississippi Basin, Mississippi river. Coagulated, lime, ferrous sulphate, filtered . . . .
S.M. 3	Mississippi Basin, Mississippi river. Coagulated, lime, ferrous sulphate, filtered . . . .
S.L. 2	Great Lakes. Coagulated (alum), filtered . . . .
S.M. 4	Mississippi Basin, Missouri river. Coagulated, lime and alum, not filtered . . . .
D.S. 1A	Eastern surface supplies, river. Untreated . . . .
S.L. 3	Great Lakes, tributary stream. Filtered. . . . .
D.M. 1A	Arizona, wells. Untreated . . . .
S.S. 1	Eastern surface supplies, river water. Fully treated . . . .
D.M. 3	Ohio, various sources, wells and surface . . . .
D.L. 2	Long Island, wells. Untreated . . . .
D.M. 2	Connecticut, surface water. Untreated . . . .
D.S. 1B	Eastern surface supplies, river. Coagulated (alum) and filtered .
D.M. 1B	Arizona wells. Untreated . . . .
S.S. 2	Eastern surface supplies, river water. Untreated . . . . .
D.M. 4	New York, mixed well and surface supplies . . . .
D.S. 4A	Eastern surface supplies, river. Coagulated (alum), filtered, lime and chlorine . . . .
D.S. 2	Eastern surface supplies, river. Untreated . . . .
D.S. 3	Eastern surface supplies, river water. Untreated . . . . .
D.L. 1	Long Island, wells. Part filtered . . . .
(p282, N.E.W. W.A. report)	An Ohio river, untreated pH value 7·9, salts 260 p.p.m. Treated lime-soda process . . . .
Thirlmere	Cumberland, moorland water. Untreated after storage in reservoir . . . .
,,	Cumberland, moorland water. Untreated after storage in reservoir . . . .

THE LAST TWO COLUMNS ARE BASED ON EQUATIONS (11B) (P. 117) RESPECTIVELY.

Water.				Age, T: years.	Average diameter, d: inches.	Williams-Hazen coefficient, W: $\frac{\text{feet}^{0.37}}{\text{seconds}}$	Chezy coefficient C: $\frac{\text{feet}^{0.5}}{\text{seconds}}$	Growth rate, a (inches per year).	A: $\left(\log \frac{\text{years}}{\text{inches}}\right)$
pH.	p.p.m.	Alkali: p.p.m.	Salts: p.p.m.						
8·3	0	83	97	30	25·8	100	98	0·0026	3·15
8·7	0	34	115	30	25·5	99	96	0·0027	3·14
9·4	—	—	—	30	28·1	96	95	0·0034	3·04
9·0	—	—	high	30	21·0	89	85	0·0053	2·84
7·5	1·6	—	—	30	30·7	90	89	0·0059	2·80
8·0	—	159	225	30	25·4	87	85	0·0067	2·74
6·9	6·0	35	—	40	7·8	55	49	0·0090	2·61
7·8	—	—	—	15	19·2	85	81	0·013	2·45
7·7	—	—	240	—	6·0	66	58	0·014	2·42
7·1	—	35	—	30	31·4	77	77	0·015	2·39
—	—	—	—	30	7·4	—	—	0·022	2·23
6·0	9·6	—	17	—	6	—	48	0·023	2·21
6·7	—	—	16	30	5·2	52	46	0·023	2·21
6·5	11·0	24	—	25	6·9	59	52	0·024	2·19
7·7	—	—	108	30	6·0	53	47	0·025	2·17
6·9	2	—	30	30	27·5	68	68	0·025	2·17
6·8	—	—	67/20	30	6·2	46	41	0·040	1·97
6·6	—	—	—	5	7·0	73	65	0·046	1·90
6·8	—	24	41	30	7·8	47	43	0·057	1·81
6·9	—	—	—	30	9·7	43	40	0·067	1·74
5·9	17·2	86	30	24	6·7	41	37·5	0·073	1·70
9·0	—	52	97	20	23·2	55	55	0·081	1·66
6·5	—	—	—	10	44	—	—	0·016	2·36
6·5	—	—	—	20	40	—	—	0·025	2·17

Fig. 6.

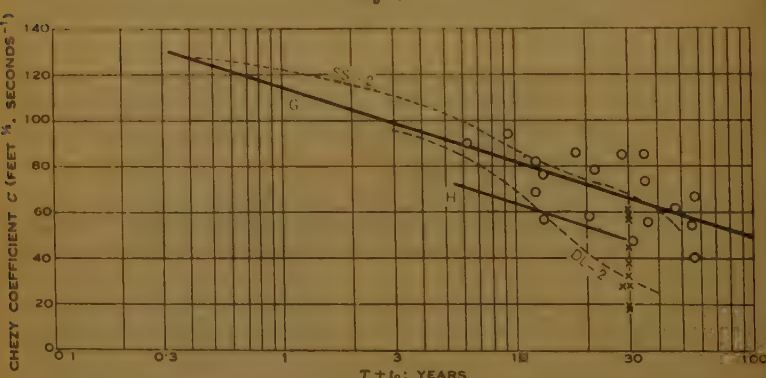


DATA AS IN *Figs. 4* and *5*, PLOTTED TO LOGARITHMIC SCALE WITH SHIFT OF TIME-ORIGIN.

(Curves D and E omitted : Curve I for Rochester 48-inch pipe ; curve J for Thirlmere 44-inch pipe.)

" after coagulation with alum, softening with lime and soda, followed by filtration," was " supersaturated with normal carbonates of calcium and magnesium, and extensive incrustation occurred," with the result that the growth-rate exceeded that given above by no less than forty times. Under more normal conditions the scatter is of the order of two or three times, implying that the pH value cannot be

Fig. 7.



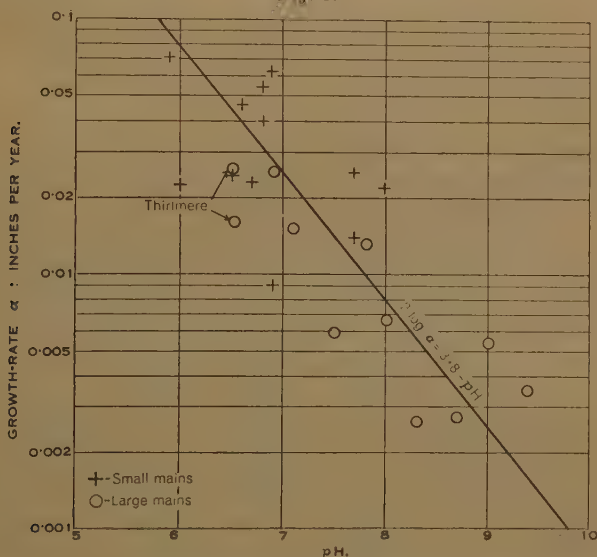
○ SS—2 : N.E.W.W.A. Committee's curve for Eastern Supplies, each point a separate pipe.

× DL—2 : N.E.W.W.A. Committee's curve for Long Island, each point a district.

Curves G and H : corresponding curves in accordance with present theory.

relied upon to predict Chezy coefficients within ± 10 units (feet$^{\frac{1}{2}}$ seconds^{-1}) and that the actual life of a pipe may be half or double that predicted. However, these uncertainties can usually be avoided, since in most districts there are records of the ageing of at least one pipe from which the local growth-rate can be found directly.

Fig. 8.



EFFECT OF ALKALINITY UPON GROWTH-RATE IN TARRED CAST-IRON PIPES.

(Points relate to data in Table I; all but two are for American districts.)

The other factor, t_0, the shift of origin as defined by equation (10), depends partly upon the initial Chezy coefficient and partly upon the growth-rate. Its value based on Table I ranges from 5 weeks to 5 years, with 8 months as the mid-value.

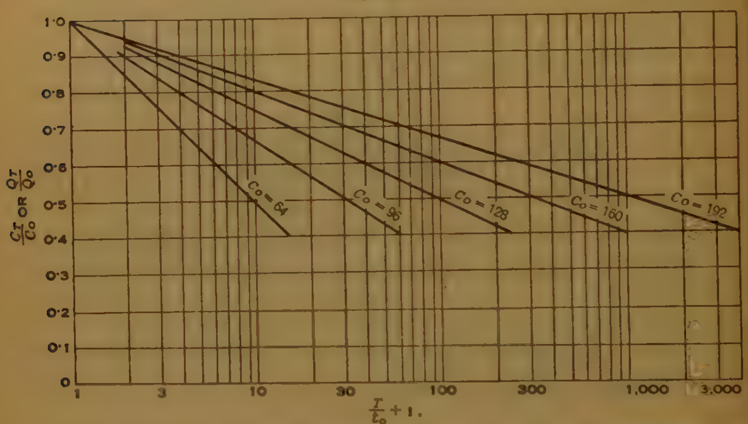
Fig. 9 has been constructed from equation (9A) to show the predicted variation of quantity with time. The abscissa is the time expressed in multiples of the shift t_0, but is readily converted to years by moving the scale bodily to the left or right as the case may be. The curves each relate to one particular initial Chezy coefficient, but are otherwise quite general, the rate of growth being completely taken into account in the value of t_0. The latter is readily obtained from *Fig. 10* or from equation (10), whichever is most convenient, and will be found to be nearly independent of d unless the pipe is initially very smooth, in which case antilog $(C_0/\sqrt{8g})$ tends to become proportional to $d^{\frac{1}{6}}$ and t_0 proportional to $d^{-\frac{1}{2}}$.

Formulas derived and Conclusions.

The two formulas (9A) and (10) plotted in *Figs. 9* and *10* are quite convenient for predicting loss of capacity. The fact that the time t in equation (9A) differs from the actual time T that the pipe has been in service causes no difficulty since the two times are connected by the simple relation

$$\frac{T}{t_0} = \frac{t}{t_0} - 1.$$

Fig. 9.



General Relation between Discharge and Age (Q_T = discharge at age T; see *Fig. 10* for t_0).

Nevertheless, there is a certain advantage in combining the two relations into a single formula conveniently written as

$$\frac{Q}{Q_0} = -\frac{1}{p_0} \log \left(\frac{T\alpha}{3{\cdot}7d} + 10^{-p_0}\right) \quad . \quad . \quad . \quad . \quad (11)$$

When, as in practice, the diameter of a proposed pipe has to be determined, equation (11) must be solved for d. There is no exact solution and the trial-and-error method is tedious. Fortunately, however, a convenient and extremely close approximate solution has been found, taking the form

$$d^{\frac{5}{2}} = \frac{1{\cdot}2Q}{\beta\sqrt{gi}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11\text{A})$$

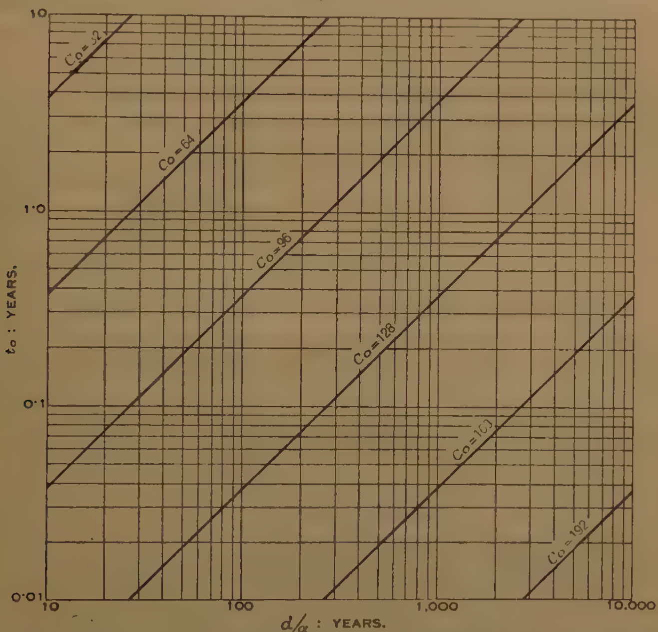
where β denotes $\log \frac{Q}{\sqrt{gi}} \frac{14}{(\alpha T + k_0)^{\frac{5}{2}}}$, Q denotes the desired flow at T, i denotes the slope, and k_0 denotes the original roughness-size, say 0·01 inch.

Equation (11) can, however, be solved for α, giving

$$\alpha = \frac{3{\cdot}7d}{T}(10^{-p} - 10^{-p_0}) \quad . \quad . \quad . \quad . \quad . \quad . \quad (11\text{B})$$

where $p = C/2\sqrt{8g}$, which is more suitable for computing α from experimental observations. If none is available then α for tarred

Fig. 10.



SHIFT OF TIME-ORIGIN t_0 AS FUNCTION OF PIPE-DIAMETER d AND GROWTH-RATE α, FOR VARIOUS TYPES OF PIPES AS DISTINGUISHED BY THE INITIAL CHEZY COEFFICIENT C_0 (FEET$^{\frac{1}{2}}$ SECONDS^{-1}).

cast-iron pipes may be estimated from the pH value of the water, using the interpolation formula

$$2 \log \alpha = 3{\cdot}8 - p\text{H} \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

which gives the growth-rate in inches per year, but this is not reliable, nor is it valid for cleaned pipes. Except for this pH equation, all equations are dimensionally complete and any consistent system of units may be used. The Chezy coefficient C_0 and the flow Q_0 relate to conditions at the beginning of the time-interval T, which need not necessarily begin when the pipe is new, and Q is the flow at the end of the period T, the gradient being assumed unchanged, while α is the average rate of growth of roughness over this interval. Hence, from the practical point of view, α can be determined from measurements with a pipe already in service,

though in order to test the formulas and to guide future development, further data are needed from pipes both in their early and in their subsequent life.

The Authors are indebted to the generosity of the Clothworkers' Company who, in supporting another research of purely academic nature, indirectly inspired the present work. It was carried out in the Civil Engineering Department of the Imperial College of Science and Technology.

The Paper is accompanied by ten sheets of drawings, from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Numerical Illustrations of Use of Equation (11).

Problem (1).—To find the diameter of tarred cast-iron pipe which will convey 5 cusecs 20 years hence with a gradient of 0·008 and pH value of 7·2, k_0 being, say, 0·01 inch.

The approx. form (11A) is convenient, $d^{\frac{5}{2}} = 1{\cdot}2\dfrac{Q}{\sqrt{gi}}\left(\log \dfrac{Q}{\sqrt{gi}}\dfrac{14}{(\alpha T + k_0)^{\frac{5}{2}}}\right)^{-1}$.

Now, from (12), $\alpha = 10^{-1\cdot 7} = 0{\cdot}02$ inch per year.

So $\alpha T + k_0 = 0{\cdot}40$ inch $+ \; 0{\cdot}01$ inch $= 0{\cdot}41$ inch $= 0{\cdot}034$ foot,

and $\dfrac{14}{(\alpha T + k_0)^{\frac{5}{2}}} = \dfrac{14}{0{\cdot}034^{\frac{5}{2}}} = 65{,}000$ feet$^{\frac{5}{2}}$

also $\dfrac{Q}{\sqrt{gi}} = \dfrac{5}{(32{\cdot}2 \times 0{\cdot}008)^{\frac{1}{2}}} = 9{\cdot}9$ feet$^{\frac{5}{2}}$.

Now $\log (65{,}000 \times 9{\cdot}9) = 5{\cdot}81$

so $d^{\frac{5}{2}} = \dfrac{1{\cdot}2 \times 9{\cdot}9}{5{\cdot}81} = 2{\cdot}05$ feet$^{\frac{5}{2}}$.

Hence $d = 1{\cdot}33$ (16 inches diameter).

Problem (2).—The Chezy coefficient of a 30-inch pipe drops from 96 in 1920 to 75 in 1935; find the growth-rate over the intervening 15 years.

Equation (11B) is convenient, $\alpha = \dfrac{3{\cdot}7d}{T}(10^{-c/32} - 10^{-c_0/32})$.

Hence $\alpha = \dfrac{3{\cdot}7 \times 30 \text{ inches}}{15 \text{ years}}(10^{-2\cdot 34} - 10^{-3})$

$= 7{\cdot}4(0{\cdot}0046 - 0{\cdot}001)$.

Hence $\alpha = 0{\cdot}027$ inch per year.

Paper No. 5080.

"The Failure of Girders under Repeated Stresses."

By Professor FREDERICK CHARLES LEA, O.B.E., D.Sc. (Eng.), M. Inst. C.E., and JOHN GWYNNE WHITMAN, M. Eng.

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

INTRODUCTION.

THE first test of the effect of repeated stresses upon riveted girders were those by Mr. (later Sir William) Fairbairn at Manchester. The girder tested by Fairbairn failed through the rivet-holes in the top flange of the girder. It is rather remarkable that further work has not been done to complete the very incomplete work of Fairbairn, and to give a definite indication of the actual safe fatigue-ranges of built-up girders. During the last 60 years a very extensive literature on the effect of repeated stresses on metals has been accumulated.[2] The behaviour of steel plates in the black condition and of plates pierced by rivet-holes, and the effect of other discontinuities such as are found in girders and other structures [3] has, however, received less attention than the subject deserves.

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[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—ACTING SEC. INST. C.E.

[2] H. J. Gough, "The Fatigue of Metals." London, 1924.

——, report on "The Present state of Knowledge of Fatigue of Metals," New International Association for the Testing of Materials, Zurich Congress, 1931.

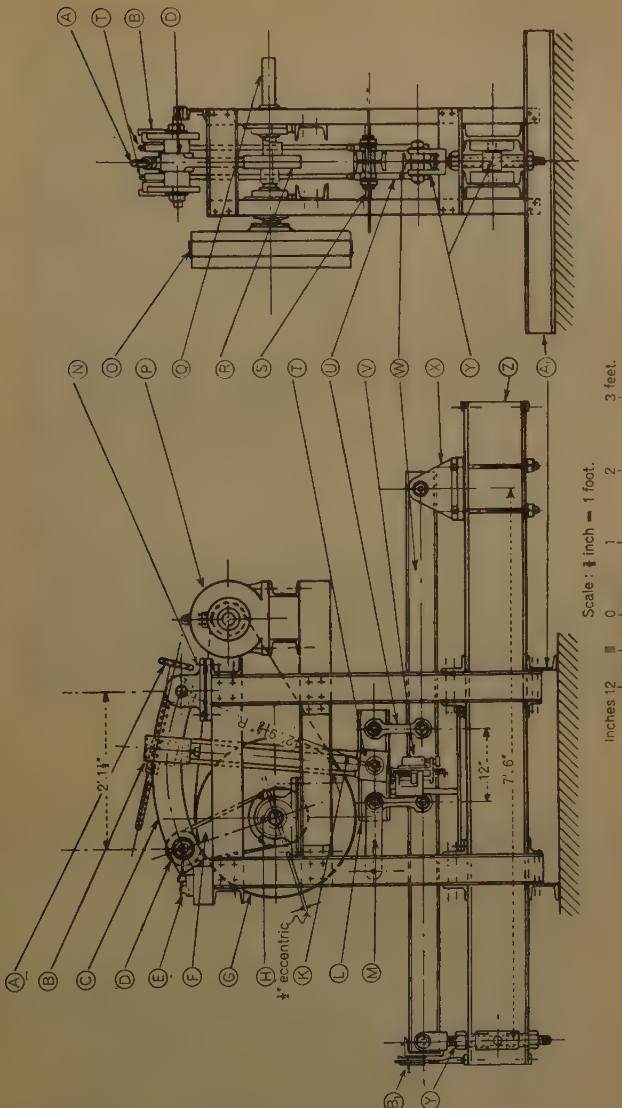
[3] Professor F. C. Lea, "Repeated Stresses on Structural Elements." Journal Inst. C.E., vol. 4 (1936–37), p. 93. (November, 1936).

The present Paper describes a testing-machine, and also some preliminary experiments that have been carried out on rolled-steel joists with drilled flanges and on butt-welded rolled-steel joists. The experiments are at present only in their preliminary stage, but they indicate the importance of further work, and already a number of riveted and welded girders are either being tested or are being constructed ready for test.

Description of Girder-Fatigue Testing-Machine.

A descriptive drawing of the fatigue-testing machine is shown in *Figs. 1*; the drive from the motor P is transmitted direct to the shaft Q carrying the flywheel G by the belt O. The flywheel is keyed to the eccentric-shaft Q which runs in two ball bearings H. An eccentric on the shaft Q has a throw of 1 inch and oscillates the connecting rod F, the big end of which envelops the ball bearing R. The little end of the connecting rod F is a plain phosphor-bronze bearing D. The movement of the rod F oscillates the movable end of the curved arm C on which can slide the crosshead B. The other end of the curved arm C pivots in phosphor-bronze bearings in the brackets N. The throw of the crosshead B is communicated to the centre of the auxiliary girder L by two pairs of connecting rods K; the ends of these house phosphor-bronze bearings T. The machine was so designed that the curved slide is an arc of a circle with its centre at the centre of the auxiliary girder L; thus the amount of movement of the auxiliary girder is proportional to the distance of the crosshead from the bearings of brackets N. The throw of the crosshead B can be varied from $\frac{1}{8}$ inch to $\frac{3}{4}$ inch approximately. The crosshead is made to slide on the curved arm C by the long screw A. The motion of the auxiliary girder is restricted practically to a vertical oscillation by the guide-links M oscillating about a fixed centre on the frame of the machine. In order to keep this oscillation as nearly vertical as possible, the centre of the bearings about which the guide-links turn is made slightly above that of the auxiliary girder when the latter is at the bottom of its movement. The movement is conveyed to the specimen W by means of the connecting links U. The bearings of the specimen are shown at X and Y; those at X are plain bronze bushes carried in two brackets bolted to a base-plate, which in turn is clamped on to the base-girders Z in any desired position. The base-girders themselves can slide in the frame so that specimens of different length can be accommodated without altering the position of the adjustable free bearing Y relative to the base-girders. Bearings Y allow for free horizontal movement and at the same time can be fixed in any position vertically. The horizontal movement allows free bending with no

Figs. 1.



Girder-Fatigue Testing Machine.

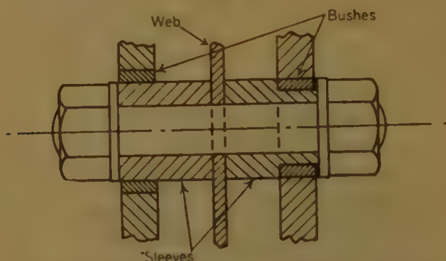
end constraint, and the vertical movement can be used for taking up creep or for setting a mean stress in the specimen, or both. The actual bearings are similar to those of X. The bearing Y consists of a fork with a screwed shank which passes through a block, to which it is clamped by two long nuts which project below and above the base-girders; the block pivots in two thick plates which are bolted to the webs of the base-girders. The left-hand end of the girder can

move freely in a horizontal plane, and vertical adjustments are made by the nuts.

Method of Attaching Specimen to the Bearings.

A hole is drilled and reamed (*Fig. 2*) in the centre of the web, and through this passes a 1-inch diameter bolt screwed at both ends. This bolt has two sleeves made of hardened steel which turn in bronze bushes. When the nuts are tightened on the screwed part of the bolt, the sleeves clamp tightly on to the girder and distribute the load to the girder-web. Thus the girder is held firmly in a vertical position whilst no restriction is offered to deflexion in the plane of the girder. One of the two sleeves is machined to a smaller diameter (*Fig. 2*) for the part of it that bears in the bush, thus checking any tendency for lateral movement of the bolt. The bearings for the connecting links U and the bottom of the connecting

Fig. 2.



METHOD OF ATTACHING SPECIMEN TO BEARINGS.

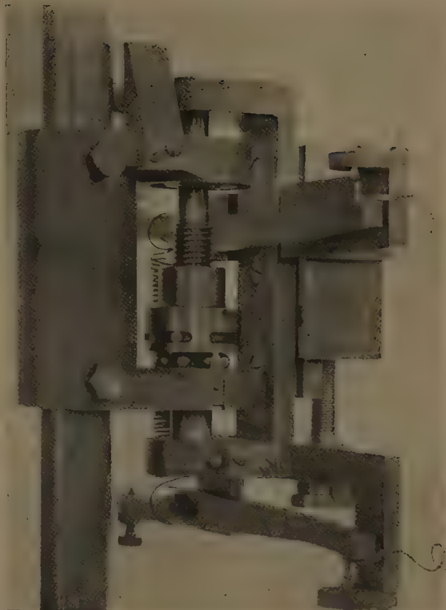
rods K (*Figs. 1*) are similar in construction except that they have two sleeves with shoulders and two washers bearing against the web. The construction of the two end bearings (*Fig. 2*) makes it easy to remove the pin and bushes. The drilling of the hole in the web reduces the moment of resistance by only 0·17 per cent. As most of the specimens to be tested have joints in the length subjected to constant bending moment, failure is not likely to occur, in these cases, at the pins connecting the links U to the girder.

Amplitude-Measuring Device.

To determine the load on the girder under test it is necessary to measure the amplitude of deflexion while oscillating at 320 cycles per minute, which is the speed at which it has been found convenient to run the machine and at which all synchronizing oscillations are avoided. The amplitude-measuring device is shown in *Fig. 3*, and it is shown in position in the girder-testing machine in *Fig. 4*. The large circular graduated nut, which turns on balls (*Fig. 3*), works on a screw which carries all the moving part of the apparatus. Thus

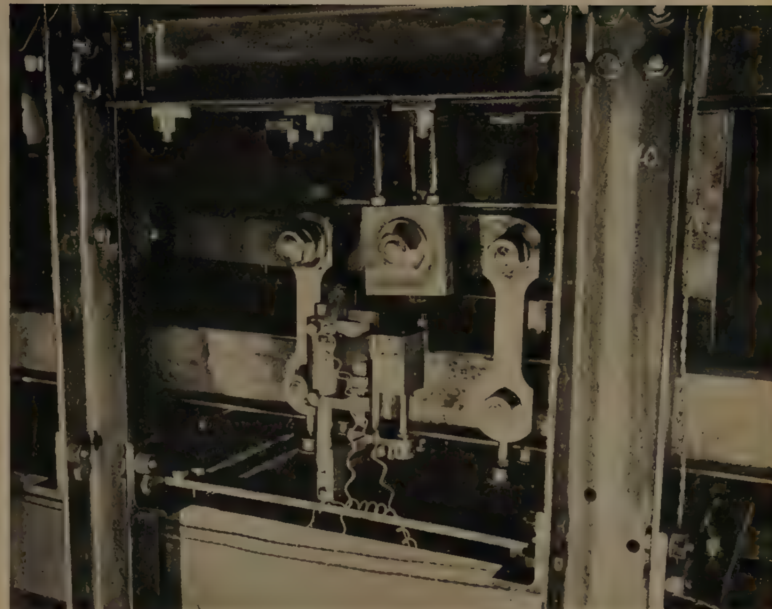
5080.

Fig. 3.



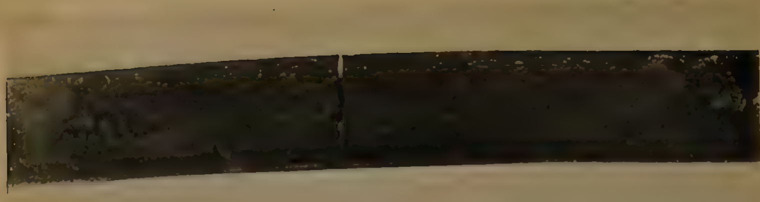
AMPLITUDE-MEASURING DEVICE.

Fig. 4.



MEASUREMENT OF AMPLITUDE.

Fig. 6.



TRIAL RUN: FRACTURE IN COMPRESSION-FLANGE (PLAN).

Fig. 7.

TRIAL RUN: FRACTURE IN COMPRESSION-FLANGE (ELEVATION).

Fig. 9.

HAIGH MACHINE TEST-SPECIMEN.

by turning this nut the two levers can be raised or lowered together without altering the span between them. The moving part of the apparatus can be swung clear of the girder through a right angle without altering the vertical height of either of the levers. A stop is provided so that the levers always swing back to the same position. The slide is moved by the micrometer-screw. This carries the top lever only, thus providing an independent movement of the top lever with respect to the bottom one. The pitch of the larger screw is 10 threads per inch whilst the finer screw has 20 threads per inch. Thus the simultaneous movement of the levers can be read to 0·001 inch and the adjustment of the span between the levers to 0·0005 inch. One end of each lever is provided with an adjustable point for making contact with the girder; at the other end is an adjustable brass contact consisting of brass screws, the fixed one of which is held in ebonite. The lever is so pivoted that any movement at the girder end is doubled at the brass-contact end. The brass screws are held in contact by a spring which can be seen in *Fig. 3*; the spring is arranged so that the same surface is in contact and slackness of the bearing will not affect the readings. The two contacts each complete a circuit including a voltmeter, and immediately a break occurs the needle swings to zero. When one of the contacts is being used a small piece of insulated packing is inserted in the other so that without altering the circuit either of the contacts may be used separately; when neither is being used both contacts are kept open in this manner. By turning either the graduated nut or the finer screw-head the lower or upper lever, respectively, is brought into contact with the test-girder, causing a break in the circuit which can be determined to the degree of accuracy of the graduations of the measuring device. When the girder is oscillating the accuracy suffers a little because the voltmeter-needle oscillates instead of swinging to zero, although this method could be relied upon to give the amplitude to 0·002 inch, which is only $\frac{1}{4}$ per cent. error when the deflection is $\frac{1}{2}$ inch. At first, in place of a resistance a pair of headphones were inserted, and the make and break were detected orally, but this method was found to give no better indication than the voltmeter.

Constant Bending.

The method of loading is so arranged that the central length of 12 inches has the same bending moment overall, and consequently no shear. This length will be convenient for testing welded joints, riveted joints, etc.

Automatic Cut-out when Girder fails.

When the girder cracks it is desirable to stop the machine. An

insulated copper strip, B_1 (*Figs. 1*), is clamped to the free-bearing end of the specimen, whilst an insulated copper point which is adjustable so as to be brought very close to the copper strip is attached to the base-girders. When the specimen cracks the adjustable free bearing Y is forced back a little, so making contact between the strip and the point; these are so wired that when contact is made the holding-on magnet of the motor-starter is shorted, and so the machine is stopped. Lately the copper contacts have been replaced with cupro-nickel alloy, which forms an oxide coating less readily than copper when subjected to electric sparking.

Bearings.

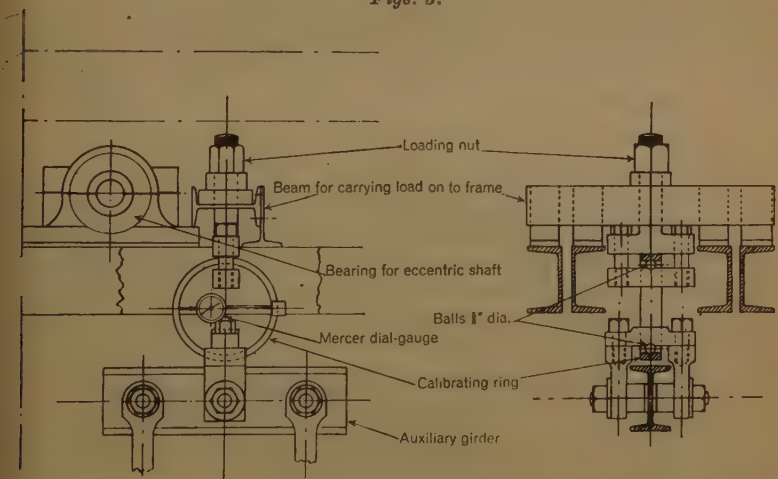
Considerable trouble was at first experienced with those bearings that had only a small angle of oscillation. Because of this small oscillation the oil-film could not be spread round the surfaces in contact, so that heavy wear resulted. Such bearings that have worn badly are the end bearings X and Y of the specimen girder (*Figs. 1*), and the top bearings of the long connecting rods T; the latter bearings had suffered most. These were split plain phosphor-bronze bearings and had a repeating pressure from zero to about 1 ton per square inch, with an angle of oscillation of 2½ degrees. When the first signs of wear were noticed drip-feed oiling was introduced, but it did not achieve its object, and wear continued, though more slowly. Ball bearings were impossible for this particular case because of the size that a load of 2½ tons would necessitate. Finally it was decided to try the experiment of replacing the existing bearings with needle-roller bearings. These consist of hardened steel rollers, something less than ⅛ inch in diameter and about 1 inch in length, taking the place of balls in a ball bearing. These needle-rollers, whilst offering a considerable bearing surface to withstand the pressure, have the additional advantage of a small diameter to facilitate lubrication with small oscillations of the bearings, and the whole assembly is comparatively small and compact. As these bearings could not be obtained split like the previous ones, the crosshead, which had been machined from a single forging, had to be cut through the centre and made to fit a sleeve which was hardened and ground on the outside to the correct diameter to act as the inner race for the rollers. The modification involved minor alterations to the other attendant parts necessitating a complete re-design of some of the parts connected with the crosshead. This bearing has given no trouble up to the present, although a considerable number of girders have been fractured since it was fitted.

Calibrating Apparatus.

Before each girder was tested under cycles of stress it was first

calibrated for load against reading of the top lever of the amplitude-measuring device. For this a special apparatus was necessary, and is shown in position in the machine in *Figs. 5*. It consists essentially of a ring of 0·45 per cent. carbon steel, heat-treated, through which the pull is transferred to the specimen. The ring deflects under the load and the deflexion is recorded on a Mercer dial-gauge, the case of which is rigidly attached to one side of the ring and the plunger of which bears against the other side. The first step was to calibrate the ring, and this was done in the Olsen testing machine. The ring, which was $7\frac{3}{4}$ inches in external diameter by $\frac{1}{2}$ inch thick and $1\frac{1}{2}$ inch wide, stood a load of $4\frac{1}{4}$ tons for 16 hours without any permanent set, and gave a straight-line graph of load against dial-reading. A short

Figs. 5.



CALIBRATING APPARATUS.

beam (*Figs. 5*) was made to take the load and transfer it to the frame of the girder-testing machine, and had a hole in the centre of the web for a $1\frac{3}{4}$-inch diameter screw with ample clearance. Loading is done by turning the nut with an 18-inch long spanner. To ensure that the ring deflects freely and in the same manner at each loading it is made to bear only on two $\frac{5}{8}$-inch balls, for which seatings $\frac{1}{16}$ inch deep are provided. The load is thus transferred to the auxiliary girder and to the specimen; the deflexion of the specimen girder is thus calibrated directly against the load in tons.

PROCEDURE EMPLOYED FOR A TEST.

The first step was to determine the section-modulus or moment of inertia of each specimen. For this a length of approximately 1 inch was cut off the end of each specimen. The two surfaces of the 1-inch

piece were ground smooth and perpendicular to the flange-surface, and the burr was carefully removed from the edges of the ground saw-cut face, which was finally polished with emery cloth. At this stage Vickers diamond-hardness values were taken on the polished face, three indentations on the web and three on each flange. Hardness-values taken on or near the surface were found to be about 10 points lower than the rest of the section. To obtain an impression of the section thus obtained the polished face was covered with printer's ink and pressed on to flimsy paper. Having obtained the section the well-known graphical method was employed to determine the second moment of area of the figure and also the section-modulus. The value of the second moment of the area quoted by the makers for a section 5 inches by 2½ inches by 9 lb. was 10·91 inch^4 units. The values obtained graphically were :—

Girder	M.S. 1	M.S. 2	M.S. 3	M.S. 4	M.S. 5
Inertia : inch^4 units	9·71	9·73	9·77	9·61	9·70

A considerable amount of work was done to provide a check for the values of the second moments thus obtained. In each of the girders tested three constructions were made for each section. It is fair to assume that the mean of three could be relied upon to be accurate to within 1 per cent. of the correct value. That the girders supplied had a section less than the ideal section is shown by the weights. These were :—

Girder.	Weight per foot : lb.	Below weight : per cent.
M.S. 5	8·42	6·4
M.S. 6	8·39	6·8
M.S. 7	8·41	6·6

The weight quoted by the makers is 9 lb. per foot.

Calibration of the Test-Girder.

Having obtained the section modulus, the girder was carefully marked out, holes being drilled on the neutral axis at 12-inch and 7-foot 6-inch centres, symmetrical about the centre (*Figs. 1*). These holes were drilled and reamed to 1 inch diameter. The test-girder was then bolted in position on the machine as seen in *Figs. 1*, and the calibrating apparatus was placed in position as shown in *Figs. 5*. The bottom lever of the amplitude-measuring device was then brought just into contact with the girder, and the same was done with the top lever. This was taken as the zero from which the top-

lever micrometer was calibrated against the Mercer dial in the calibrating ring.

The load was applied through the loading nut and the reading of the top-lever micrometer was taken for every 10 divisions of the dial gauge up to 90. This corresponds to a load of 2·85 tons and a stress of 14 tons per square inch. The loading was repeated until no permanent set resulted, and a graph was plotted of the last set of readings. The calibrating apparatus was then removed and the connecting rods were replaced in position; any alteration in the vertical position of the test girder that was necessary to replace the connecting rods was done with the adjustable free bearing Y (*Figs. 1*). The bottom lever of the calibrating apparatus was then brought up to the specimen so that the same part of the top micrometer screw was used for setting the deflexion as for the calibration.

Having obtained the straight-line calibration-graph, the deflexion required for a given stress was calculated as follows:

Example taken: girder M.S. 3. Stress required = 18 tons per square inch.

Modulus value Z = mean of 3·920, 3·933, 3·933 = 3·93 inch^3 units.

From the calibration-graph, dial-reading 90 gives a movement at the centre of the girder of 6·68 turns of the top-lever micrometer, and from the calibration of the ring dial-reading 90 is equivalent to 2·85 tons. The loading is as shown in *Figs. 1.* Then, for a stress of 18 tons per square inch,

$$\text{Bending moment} = \frac{W}{2} \times 39 = f.Z.$$

Hence
$$\frac{W}{2} \times 39 = 18 \times 3{\cdot}93$$
$$\text{and } W = 3{\cdot}627 \text{ tons.}$$

Number of turns of top lever micrometer required for 3·627 tons load $= 6{\cdot}68 \times \dfrac{3{\cdot}627}{2{\cdot}85} = 8{\cdot}50$ turns.

Hence 8·50 turns of the top lever micrometer give a maximum fibre-stress of 18 tons per square inch.

Fixing the Range of Stress for the Specimen Girder.

The zero position of the top lever micrometer was first found by gradually bringing the bottom lever up to the girder while slowly oscillating the flywheel by hand about the bottom position until contact was made. By this means the bottom position of the throw of the girder was obtained and then the top lever was brought down until it was just touching the top flange of the girder; the slack of

the bearing of the top-lever micrometer-head was thus always in one direction, and the top lever had the same initial reading as that of the set of readings with which the girder was calibrated. All measurements with either micrometer were taken using the break of the circuit.

Having found the initial position, 8·50 turns were added on to this reading. Thus, to obtain the required range of stress, the girder must so oscillate between the two levers that it just touches both. The motor was started and the crosshead was gradually moved along the slides by the long screw A (*Figs. 1*). This gradually increases the amplitude but without altering the lowest point of the oscillation. The amplitude was, therefore, increased in this manner until the girder was just touching the top lever. The crosshead was then clamped in position, any reversed load was taken up with the adjustable bearing Y (*Figs. 1*), and finally the amplitude was again measured. If the measurements were different from the above 8·50 turns, being, say, 8·53 turns, the necessary alteration would be made to the calculated range of stress.

Should, at any time during the run, a check be required on the amplitude, the bottom lever must be brought up until it just touches the girder and then the top lever should be just touching also.

Accuracy.

Taking into account the probable errors due to determination of section-modulus, calibration, measurement of amplitude, and effects due to errors of alignment, it has been estimated that the total possible error could not be more than from 2 to 2½ per cent. or, say, between ¼ and ½ ton per square inch in a range of stress of 15 tons per square inch.

Trial Run.

The first test was with a girder 4¾ inches by 1¾ inch, a heavy range of 26 tons per square inch being imposed with a minimum stress of zero. The girder immediately began to creep quickly in the direction of loading, and over 1 inch was taken up by the adjustable free bearing, but it was not sufficient and the machine was run with a considerable amount of reversed load.

The method employed for determining whether there was any reversed load was to slack off the top nut; any reversed load was immediately made apparent by hammering of the bottom nut against the block. The reversed load was taken up by lightly tightening the bottom nut until all tendency to hammer had ceased. The top nut was then tightened. It was found that slight creep could be detected by feeling the long connecting rods, in which any reversed load was

made apparent by a slight shock which could be identified after some practice.

After the machine had been allowed to run for 9,000 repetitions the bottom flange was noticed to be buckled, and after 36,000 repetitions a crack appeared in the centre of the constant-bending portion in the compression-flange (*Figs. 6* and *7*, facing p. 123). After another 10,000 reversals the machine was stopped, the girder having nearly cracked through although there was no sign of a crack in the tension-flange.

The curious fact that a girder should break by fatigue in the compression-flange required some explanation, and a rough calculation was made to determine the nature of the stresses due to the buckling. The buckled flange is shown in *Fig. 6*. The eccentricity at the centre as near as could be estimated was of the order of $\frac{1}{8}$ inch. Let the total load in the flange be represented by P tons. Then the bending moment at the centre due to buckling $= \frac{P}{8}$ inch-tons. The maximum stress due to combined bending and compression $= \frac{P}{A} \pm \frac{My}{I}$, approximately.

Assuming a flange-thickness of t inches, with a width $= 1{\cdot}75$ inch, then the maximum stress on the inside of the flange $= \frac{P}{A} + \left(\frac{P}{8} \times \frac{y}{I}\right)$. But $\frac{I}{y} = \frac{t \times 1{\cdot}75^2}{6}$, and $P = 26 \times t \times 1{\cdot}75$ tons.

$$\text{Hence the maximum stress} = 26 + \frac{26 \times t \times 1{\cdot}75 \times 6}{8 \times t \times 1{\cdot}75^2}$$

$$= 26 + 11 = 37 \text{ tons per square inch}$$

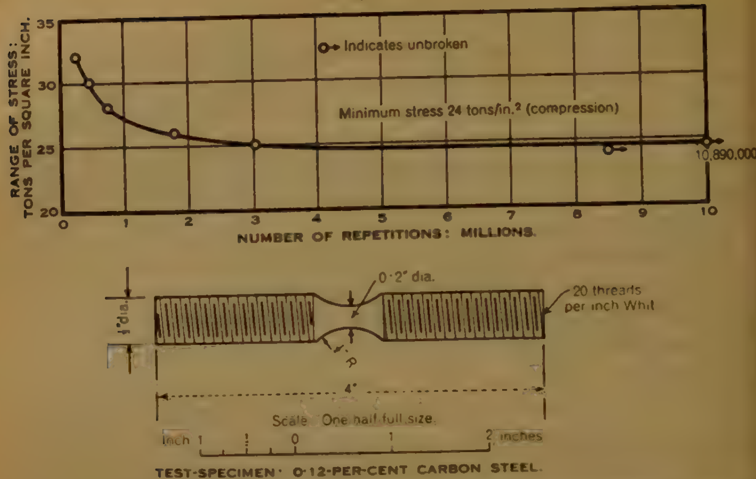
(compressive), and on the outside of the buckled flange the maximum stress $= 15$ tons per square inch (compressive).

Assuming that there was a reversal of stress of 5 tons per square inch due to reversed load, the range at the inside of the buckling is from 32 tons per square inch compressive to 5 tons per square inch tensile. The above calculation no doubt over-estimates the stresses.

To establish the fact that such a range of stress would cause a failure, a number of repeated-stress tests on specimens of 0·12 per cent. carbon steel under compression were carried out in a Haigh machine. These tests were run with a minimum compressive stress of 24 tons per square inch and the ranges of stress shown in Table I (p. 130). The results are shown in Table I and *Figs. 8*.

A fatigue range of $24\frac{1}{2}$ tons per square inch was obtained for 10^7 reversals; that is, from a tension of $\frac{1}{2}$ ton per square inch to a compression of 24 tons per square inch, which was well below the estimated stress in the compression-flange of the test girder. The

Figs. 8.



type of fracture obtained is shown in *Fig. 9* (facing p. 123). It will be noticed that neither in the circular specimens nor in the girder (*Fig. 6*) is there any suggestion of a fracture at 45 degrees to the axis.

Table I.—Compressive Fatigue-Tests on Steel A.S. 5., 0·3 inch Diameter, in the Haigh Machine.

Range of stress: tons per square inch.	Compressive mean stress: tons per square inch.	Compressive minimum stress: tons per square inch.	Number of repetitions.	Remarks.
24	12	24	8,460,000	Unbroken. Slow continuous creep for the first few millions. Final diameter increased by 0·016 inch.
32	8	24	240,000	Broken, a continuous creep.
30	9	24	450,000	Broken, a continuous creep.
28	10	24	736,000	Broken, a continuous creep.
26	11	24	1,768,000	Broken, a continuous creep.
25	11·5	24	3,038,000	Broken, a continuous creep.
24·5	11·75	24	10,890,000	Unbroken. Slow continuous creep for the first few millions. Final diameter increased by 0·015 inch.

Fatigue limit = $24\frac{1}{2}$ tons per square inch range at 24 tons per square inch minimum stress for 10^7 repetitions.

Fatigue-Tests on Mild-Steel Girders.

The girders chosen for the above tests were all of a broader-flange type than the girder used for the trial run and all failed in the tension-flange, no buckling occurring in the compression-flange. The girders were 5 inches by $2\frac{1}{2}$ inches by 9 lb., and were stamped M.S. 1, M.S. 2, etc.

The first test was on girder M.S. 1 at a stress of 23 tons per square inch. Creep occurred so quickly that it could not all be taken up, and the test was run with a slight reversed load. Fracture occurred over the left-hand bearing as seen in *Figs. 1* (p. 121), starting in the top (tension-) flange, working down into the hole in the web and then starting again from the bottom of the hole. The crack started within the constant bending portion.

The second joist, M.S. 2, was tested at a range of stress of 20·9 tons per square inch ; there was a fair amount of creep and hence a small amount of reversed load. The fracture occurred at the section through the pin-hole where the constant bending moment commences, and extended right through the girder. The point at which the crack commenced was at the inside edges of the tension-flange, whence the fracture spread through the bearing hole to the compression-flange.

Girder M.S. 3 was tested at a stress of 18 tons per square inch. There was a slight continuous creep, but it could easily be taken up and the machine was run without any reversed load. The machine was stopped before the fracture had spread far. The fracture started in the tension-flange at approximately 45 degrees to the length of the girder, and worked its way down towards, but stopped short of, the hole. In this particular case the surface of the tension-flange was filed and polished on either side of the crack, and a series of hardness-values (*Fig. 10*, facing p. 132) were taken at every 0·1 inch along the centre of the flange. The readings are shown plotted in *Fig. 11*, p. 132. There is evidence of slight work-hardening near the crack, and the constant-bending portion is shown to be slightly more work-hardened than the other part of the flange, the respective mean hardnesses being 113 and 119. This particular test gave a fracture after 811,000 repetitions, so the stress was still considerably above the fatigue-limit for more than 10^7 repetitions.

Girder M.S.4 was tested at a stress of $15\frac{1}{2}$ tons per square inch. There was still a creep, but a very slight one, and the test was run without any reversed load. The fracture, which occurred after 3,000,000 repetitions, took place 1 inch outside the constant-bending portion. The reason for this became apparent on inspection of the fracture, which revealed a scale-inclusion in the tension-flange at the point where the fracture started. This is shown by the arrow in

Fig. 11.

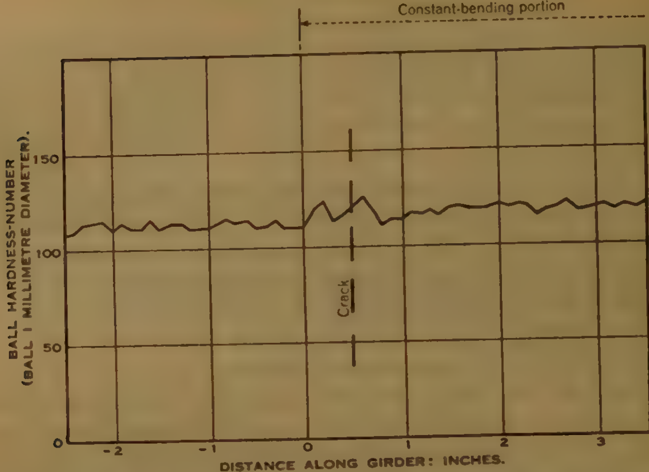


Fig. 12, which also shows very clearly the blackened part of the fracture, probably scale-dust, indicating the start of the fracture, and also the line at the bottom of the section on which the girder hinges before finally snapping. The actual breaking stress, on recalculating for the point of fracture, was found to be just above 15 tons per square inch.

As this was obviously near the fatigue-limit the next test, on girder M.S. 5, was run at 15 tons per square inch. There was not more than 0·01 inch creep, and the girder finally broke after 9,000,000 repetitions.

The results of these five tests are shown in Table II (p. 133) and *Fig. 13*, from which the fatigue-limit for 10^7 repetitions of stress, from a minimum stress of zero, is seen to be 14·75 tons per square inch.

Fig. 13.

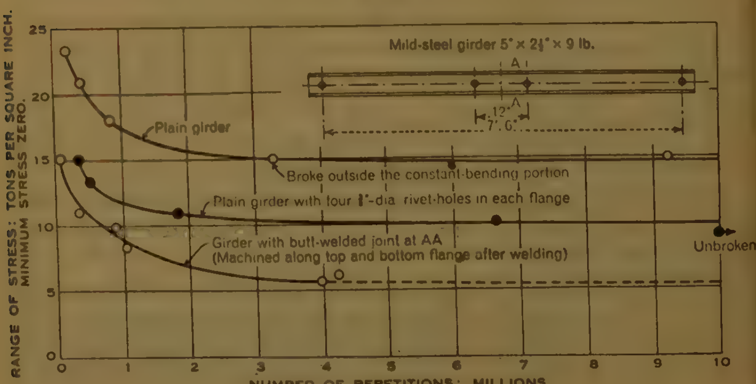
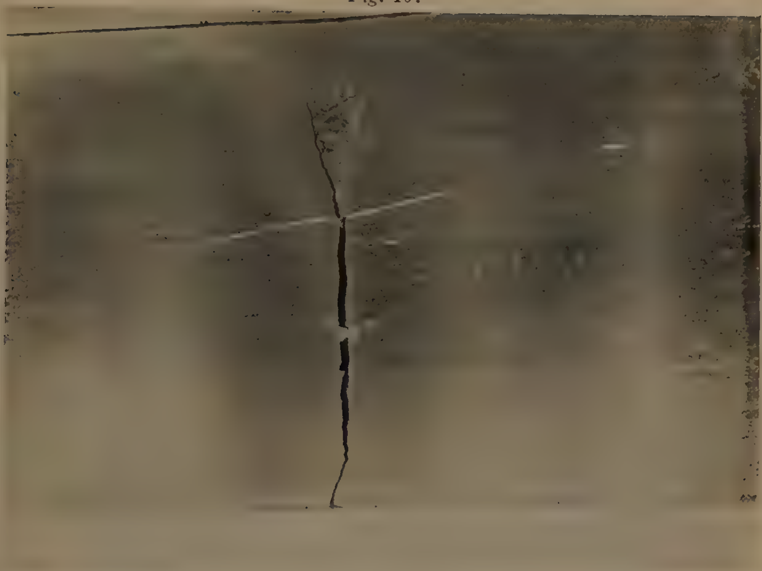
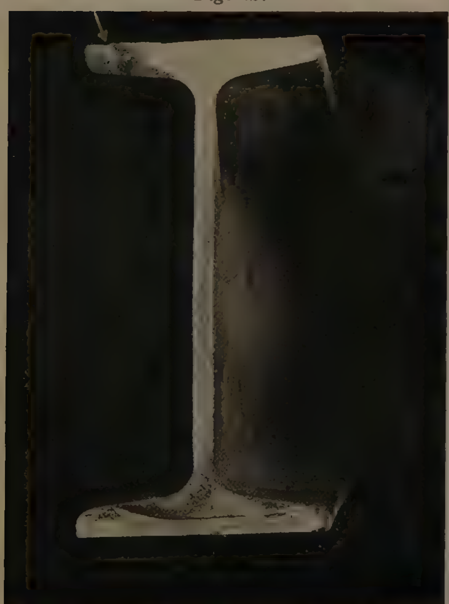


Fig. 10.



FRACTURE IN GIRDER M.S. 3.

Fig. 12.



FRACTURE IN GIRDER M.S. 4.

5080.

Fig. 22.

Fig. 23.

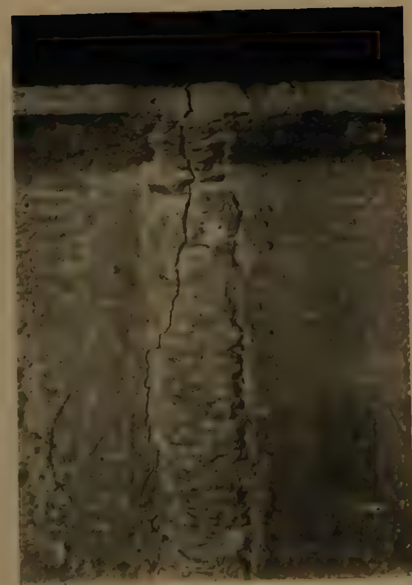
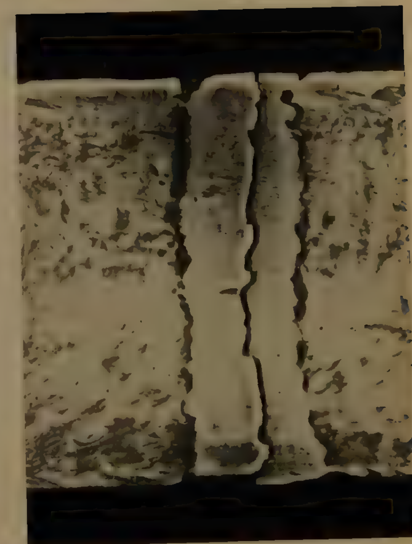


Fig. 24.



FRACTURE OF BUTT-WELDED GIRDER

TABLE II.—TESTS OF MILD-STEEL GIRDERS 5 INCHES BY $2\frac{1}{2}$ INCHES BY 9 LB., IN GIRDER FATIGUE-TESTING MACHINE.

Specimen.	Vickers diamond hardness-number.		Inertia: inch^4 units.	Range of stress: tons per square inch.	Repetitions.	Remarks.
	Web.	flange.				
M.S. 1 .	141	145	9·71	23·3	176,900	Broken. Slight reversed load was unavoidable due to quick creeping.
M.S. 2 .	147	148	9·73	20·9	352,900	Broken. Slight reversed load was unavoidable due to quick creeping.
M.S. 3 .	150	145	9·77	18·0	811,000	Broken. No reversed load,as only a slight creep throughout test.
M.S. 4 .	132	129	9·61	15·02	3,259,300	Broken outside constant bending portion. Scale inclusion at fracture. Slight creep throughout test.
M.S. 5 .	141	143	9·70	15·00	9,101,100	Broken. Very slight though continuous creep.

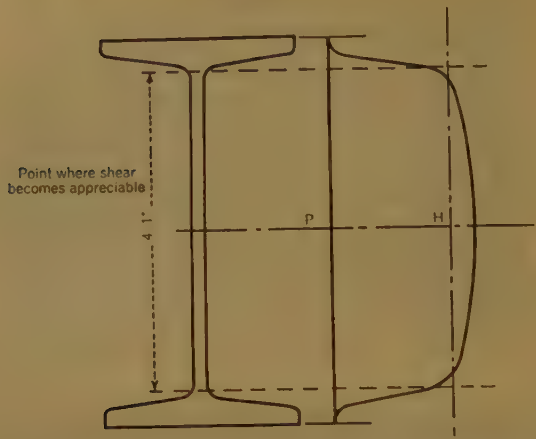
Fatigue-limit from zero minimum stress for more than 10^7 repetitions = $14\frac{3}{4}$ tons per square inch.

NOTE ON THE FRACTURES.

Four out of the five failures occurred over one or other of the bearing points where the bending moment reaches its maximum value and where the section is slightly diminished by the hole in the web, and the fifth had a scale inclusion at the point of fracture. Theoretically, between the two points of bearing the bending moment is a maximum and constant, and the shear is zero. At the section immediately to the right of the hole there is a shear equal to half the load on the girder. The distribution of shear stress over the section of the joist is shown in *Fig. 14* (p. 134). In the case of M.S. 3 the load equalled 3·6 tons and the maximum tensile stress is 18 tons per square inch. The shear stress at the top of the web is about 1·9 ton per square inch. The bending stress at this point $= \frac{18 \times 2{\cdot}05}{2{\cdot}45} = 15$ tons per square inch, and the maximum principal stress is about 15·2 tons per square inch.

Owing to creep and the consequent more even distribution of the bending stress in the flange when under load, as shown by the dotted

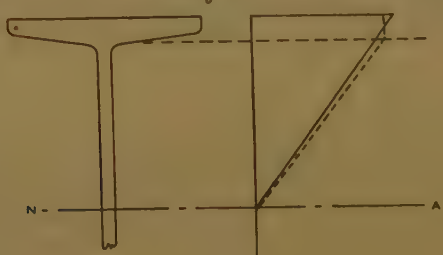
Fig. 14.



line in *Fig. 15*, it is possible that, in girders which have stresses above the yield-point of 17·5 tons per square inch, the principal stress might have a slightly larger value than the skin-stress, but this is very unlikely for stresses less than the yield-point; also, the evidence of the fractures all indicates that the crack first started in the extreme fibre, in which case the fractures were not influenced by the shear values at these sections.

The hole drilled in the web, as has been previously mentioned,

Fig. 15.



reduces the second moment of the section by 0·17 per cent., and it is thought that this reduction, small though it is, is probably the reason for failure continually occurring over the bearing points. If necessary there will be no difficulty in strengthening the section at this point, but this is not necessary for tests of girders with drilled flanges or for welded girders, as these all fracture inside the 12-inch length subjected to constant moment. Three of the five joists broke over the left-hand bearing (as the machine is seen in *Fig. 1*), one over the right-hand bearing, and one 1½ inch to the right of this bearing.

Tensile Tests of Specimens from Joists.

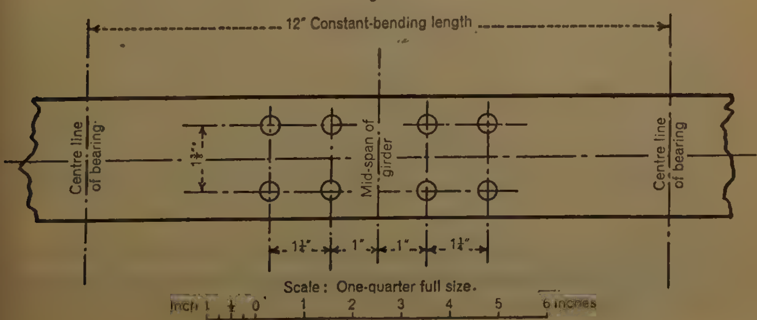
Tensile-test pieces were taken from two of the girders, two specimens from M.S. 2 and two from M.S. 4. A flat piece was cut to standard size with an 8-inch gauge-length from the web of each girder and was left black. Machined specimens were also taken out of the flanges at the end of the girder where there was least work-hardening. These were round specimens $\frac{1}{2}$ inch diameter and of 2 inches gauge-length. The results of the tests are as follows:

Girder.		Yield-point: tons per square inch.	Ultimate strength: tons per square inch.	Total extension: per cent.	Reduction of area: per cent.
M.S. 2	Flat . . .	17·76	27·1	34·8	60·6
	Round . .	17·67	27·5	38·0	66·5
M.S. 4	Flat . . .	17·64	29·9	30·7	58·6
	Round . .	17·96	27·5	42·0	65·0
Specification . . .		—	28 to 33	not less than 20	not less than 40

Bending-Fatigue Tests on Mild-Steel Girders with Rivet-Holes in the Flanges.

On the assumption that the girder was cut in two, a butt-riveted joint was designed for joining half-girders together. The size and position of the rivet-holes were then made the same as if the specimen girder were cut through the centre and the two halves were to be joined by cover-straps (*Fig. 16*). The size and distance of the rivet-holes from the edges of the flange were taken from the Dorman, Long & Co. Handbook. To ensure that these holes were identical they were drilled and reamed to the required diameter. The depth of each of

Fig. 16.



the holes at the outside and inside edges was measured with a pointed micrometer and the girder was then bolted in position in the girder-testing machine and was calibrated and tested as previously described. The moment of inertia of the whole section was determined as before, and from this was subtracted the moment of inertia of the rivet-holes, of which there were four, about the neutral axis. The thickness of the flange not being uniform means that the sections

TABLE III.—FATIGUE-TESTS IN THE GIRDER-TESTING MACHINE OF MILD-STEEL GIRDERS WITH RIVET-HOLES IN THE FLANGES.

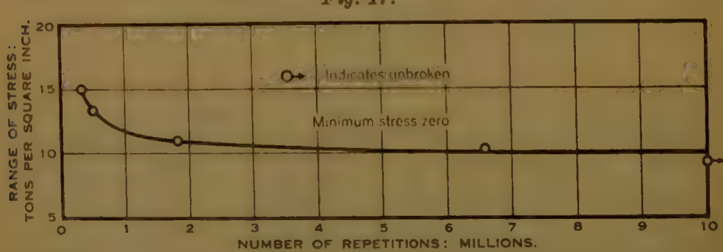
Specimen.	Moment of inertia : inch^4 units.		Vickers diamond hardness-number.		Range of stress: tons per square inch.	Repetitions.	Remarks.
	Complete section.	Net section.	Web.	Flange.			
M.S. 6 .	9·74	7·15	137	140	15·0	333,000	Broken. Slight initial creep.
M.S. 7 .	9·75	7·18	126	115	13·3	500,000	Broken. No creep.
M.S. 8 .	9·63	7·03	135	121	11·1	1,818,000	Broken. No creep.
M.S. 9 .	9·79	7·22	141	129	9·2	10,000,000	Unbroken.
M.S. 10 .	9·59	7·06	141	127	10·25	6,592,000	Broken. No creep.

Minimum stress zero.
Fatigue-limit from zero minimum stress for more than 10^7 repetitions = 10 tons per square inch.

of the holes are trapeziums and not rectangles. The error in the second moment of the area assuming the sections of the holes to be rectangles is, however, quite negligible.

The results of tests of five joists are shown in Table III and in *Figs. 13* (p. 132) and *17*. The girders broke across a section containing

Fig. 17.



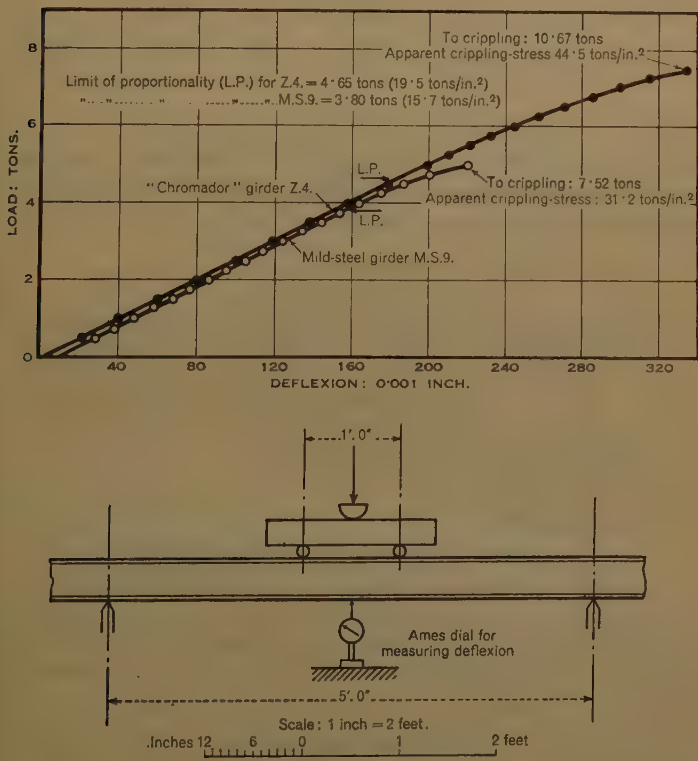
the rivet-holes and the crack invariably started at the hole-boundary. The introduction of rivet-holes in the flange of the girder reduces the fatigue-limit, from zero minimum stress, for more than 10^7 repetitions

from $14\frac{3}{4}$ to 10 tons per square inch, which is approximately a reduction of one-third; that is to say, the girder is 50 per cent. stronger without than with the holes. Thus, if cover-plates were added to a plain girder with the intention of increasing the loading capacity by 50 per cent., the net result due to the introduction of rivet-holes would probably be that the fatigue-strength would remain approximately the same.

Static Test of Mild-Steel Joist.

One of the joists, M.S. 9, was loaded to destruction in a testing-machine, as shown in *Figs. 18.* From the load–deflexion diagram

Figs. 18.



METHOD OF APPLYING LOAD.

it is seen that the stress at the limit of proportionality is 15·7 tons per square inch, which is slightly greater than the fatigue-range at zero minimum stress.

Tests of Girders of "Chromador" Steel.

A number of girders were kindly supplied by Messrs. Dorman, Long & Company, Ltd. The results of tests of specimens cut from the girders are shown in Table IV. A static test of one of the girders was also made, as shown in *Figs. 18* (p. 137). The limit of proportionality was about 19·5 tons per square inch, and the "apparent stress" at failure was 44·5 tons per square inch.

Table IV.—Tensile Tests of "Chromador" Steel cut from Rolled Joists

Flat specimens : black surface, 8-inch gauge-length.
Round specimens : polished surface, 2-inch gauge-length.

Girder specimen.	Yield-point : tons per square inch.	Ultimate strength : tons per square inch.	Total extension : per cent.	Reduction of area : per cent.	Pyramid diamond hardness.
Z. 2 Flat . . .	25·85	34·4	23·6	52·1	177
Z. 2 Round . .	24·25	36·7	34·5	71·6	182
Z. 3 Flat . . .	26·45	39·1	17·8	29·6	194
Z. 3 Round . .	24·20	41·0	30·0	70·7	213

Table V.—Fatigue-Tests on "Chromador" Girders.
(Rivet-holes in the flanges.)

Girder.	Vickers diamond hardness-number.		Moment of inertia : inch^4 units.		Range of stress : tons per square inch.	Repetitions.	Remarks.
	Web.	Flange.	Complete section.	Net section.			
Z. 1 .	184	189	10·06	7·51	15	774,000	Broken. Slight initial creep.
Z. 2 .	175	188	9·96	7·39	13·7	3,227,000	Broken.
Z. 3 .	206	211	9·99	7·41	12·8	768,000	Broken.
Z. 4 .	200	202	9·73	7·18	12·6	8,253,000	Unbroken.
Z. 5 .	186	192	10·14	7·58	13·0	10,312,000	Unbroken.

Fatigue-limit from zero minimum stress for more than 10^7 repetitions = 13 tons per square inch.

Fatigue-tests, similar to those on mild-steel joists, were carried out on "Chromador" joists with and without rivet-holes drilled in the flanges. The results of the tests are shown in Tables V and VI and in *Fig. 19*. The fatigue-range for 10^7 repetitions, at zero minimum stress, of the girder with holes in the flanges is 13 tons per square inch and is thus, for "Chromador" joists with holes in the

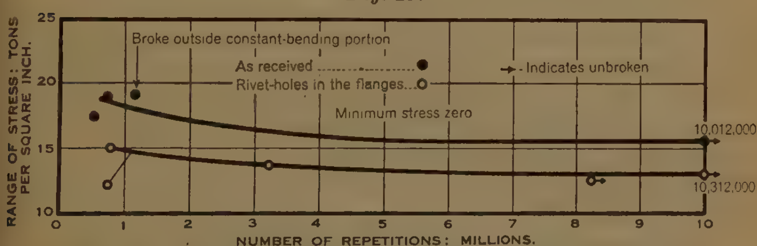
Table VI.—Fatigue-Tests on Chromador Girders in the "As Received" Condition.

Girder.	Vickers diamond hardness-number.		Moment of inertia of section: inch^4 units.	Range of stress: tons per square inch.	Repetitions.	Remarks.
	Web.	Flange.				
Z. 6 . .	201	202	9·82	19·25	1,180,000	Broken 2 inches outside constant-bending portion.
Z. 7 . .	191	186	9·87	19·1	719,000	Broken inside constant-bending portion.
Z. 8 . .	214	181	10·04	17·4	525,000	Broken inside constant-bending portion.
Z. 9 . .	179	194	9·94	15·5	10,012,000	Unbroken.

Fatigue-limit from zero minimum stress for more than 10^7 repetitions = 15·5 tons per square inch.

flanges, 30 per cent. greater than for mild-steel joists with holes in the flanges. The results of the tests of the girders without holes in the flanges are somewhat erratic. One fracture took place well outside the length subjected to the constant maximum bending moment, and taking a curve through the lowest values the fatigue-

Fig. 19.



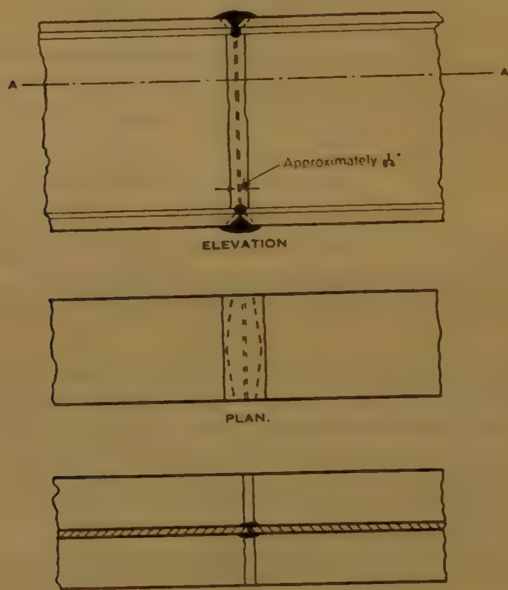
range is not much greater than for the drilled girders. From Table IV it will be seen that the results of tensile tests are variable, and this is shown in *Fig. 19*, especially in the plotted points for the girders without holes. The crack always commenced at the boundaries of the holes and usually extended about one-third of the way down the web.

Fatigue-Tests on Butt-Welded Mild-Steel Joists.

The joists, 5 inches by $2\frac{1}{2}$ inches by 9 lb., were supplied in 8-foot lengths. These were cut through the centre and the two half-girders were butt-welded together as shown in *Figs. 20*. The weld-metal

was machined level with the surface of the flanges, but that under the flange and on the web was left as received. Gauges 10 and 12 of a well-known electrode were used to make the welds, which were made in the works. Specimens were machined from the compression-flanges of girders M.S. 11 and M.S. 13, so as to include the weld.

Fig. 20.



BUTT-WELDED JOINT.

The specimen from M.S. 11 was machined to ½ inch diameter. The surface of the specimen was well below the scale.

The results of a tensile test of a ½-inch diameter specimen were :—

Yield-point : 11·5 tons per square inch.

Ultimate strength : 20·8 tons per square inch.

Total extension on length of 2 inches (including weld) : approximately 5 per cent.

The specimen taken from the girder M.S. 13 was machined until all traces of the "lack of penetration" had been removed. The diameter was ¼ inch.

The results of a tensile test of a ¼-inch diameter specimen were :—

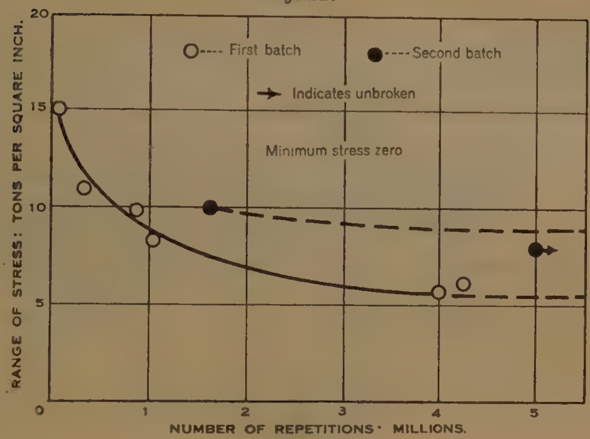
Yield-point : 18·3 tons per square inch.

Ultimate strength : 29·3 tons per square inch.

Total extension on length of 2 inches (including weld) : approximately 9·5 per cent.

Both specimens broke through the centre of the weld.

Fig. 21.



The results of the repeated-stress tests of the welded joists (Table VII and *Figs. 13* (p. 132) and *21*) are startlingly low. The fracture of one of the girders is shown in *Figs. 22, 23*, and *24* (facing p. 133). The crack started near to the centre of the weld in the tension-flange (*Fig. 22*), and proceeded partly in the weld-metal and partly in the plate. It will be seen from *Fig. 22* that near the top of the web there was evidently very poor fusion of the weld-metal and the flange. It may be that the crack commenced here. It has already been stated that these welds were made of a well-known electrode and in a works by an experienced welder. They are given here not as an example of the best that can be done by welding but rather as an example of what is to be avoided. A very large number of tests in

TABLE VII.—FATIGUE-TESTS OF WELDED JOISTS.

Girder.	Moment of inertia of section of girder : inch⁴ units.	Range of stress : tons per square inch.	Repetitions.	Remarks.
M.S. 11 .	11·27	15·1	66,000	Broken.
M.S. 13 .	11·36	11·0	310,000	Broken.
M.S. 14 .	10·72	9·9	878,000	Broken.
M.S. 15 .	10·68	8·4	1,022,000	Broken.
M.S. 16 .	10·81	6·2	4,209,000	Broken.*
M.S. 12 .	11·07	5·7	3,981,000	Broken.

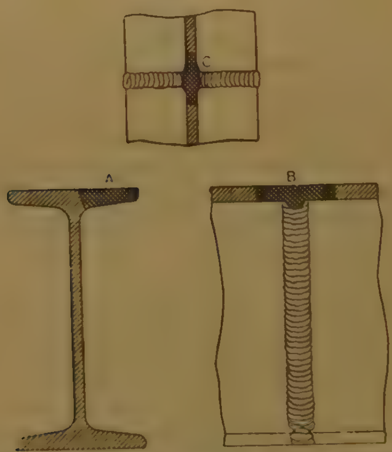
* The cut-out stopped the machine when the only sign of a fracture was an extremely fine line on the surface of the weld which did not open out at the top of the stroke ; 4 hours after restarting, the girder had cracked half-way through.

Fatigue-limit from zero minimum stress for more than 4×10^6 repetitions = 5·5 tons per square inch.

this and other machines has shown that under repeated stresses the greatest danger is lack of penetration. Other tests are in hand.

Figs. 25, 26, and 27 show enlarged sections of the welded flange taken from the positions shown in *Figs.* 28. A very large part of the weld appears to be of a fine structure. The top left-hand corner (*Fig.* 26), at the junction of the weld and the plate, has a coarse structure showing that the top layer of weld which has been machined away has not refined this part of the weld-metal. The projection on the lower part of the weld is a run of metal along the bottom of the V under the flange: between this and the V, *Figs.* 22, 25, and 26, there is very definite evidence of lack of penetration of the weld-

Figs. 28.



POSITIONS OF MACRO-SPECIMENS SHOWN IN *Figs.* 25, 26, AND 27.

metal. *Figs.* 29 and 30 (facing p. 143) show photo-micrographs of the weld-metal, taken from the upper and lower layers respectively, and also the analysis of the weld-metal. It seems only possible to account for the very low fatigue-range obtained from these girders by assuming that the fatigue-crack commenced in the neighbourhood of the area just above the top of the web, where there was a definite discontinuity and where the direct stress was not very much less than the direct stress at the outer boundary. The two small black areas, *Fig.* 26, are evidence of some type of inclusion which has not been defined.

The tests of the joists, of which particulars are given in *Fig.* 21, having given disappointingly low results, a further series was prepared by another expert welder, care being taken to insure that there was penetration at the bottom of the V. The results of the tests of these joists are also shown on *Fig.* 21 (p. 141). One specimen broke

Fig. 25.

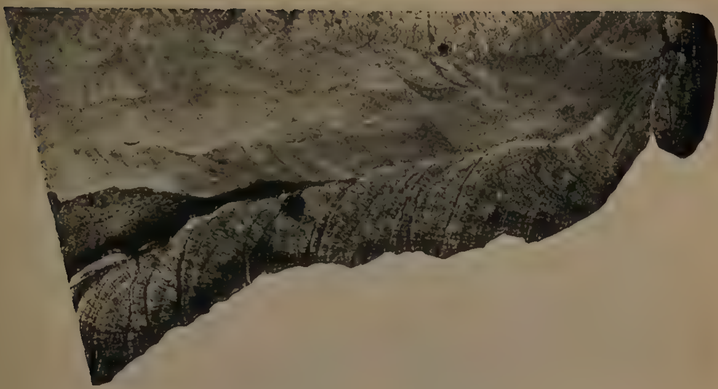


PHOTO-MACROGRAPH OF SECTION A (*Figs. 28*). ×3.

Fig. 26.

PHOTO-MACROGRAPH OF SECTION B (*Figs. 28*). ×3.

Fig. 27.

PHOTO-MACROGRAPH OF SECTION C (*Figs. 28*). ×3.

Fig. 29.

PHOTO-MICROGRAPH OF WELD-METAL. × 250.

Fig. 30.

PHOTO-MICROGRAPH OF WELD-METAL. × 250.

Analysis of Weld-Metal:

Carbon	0·11 per cent.
Manganese . . .	0·88 per cent.
Silicon	Trace.

after 1,500,000 repetitions at a stress of 10 tons per square inch. The fatigue-limit from zero minimum stress for more than 5×10^6 repetitions is approximately 9 tons per square inch. The point W in *Fig. 31* indicates the fatigue-range for these joists. Thus by care in welding the range has been increased from 5½ to 9 tons per square inch; this is slightly better than the worst result obtained from certain riveted joints tested in tension.[1]

One of the Authors has carried out, on a specially-designed machine, a large number of tests of butt-welds[2], which have given fatigue-ranges at zero mean stress of more than 75 per cent. of the fatigue-range of plates of the same tensile strength. Some specimens failed in the plate and not in the weld. Others, however, have given fatigue-ranges less than 50 per cent. of that of the plate.

Future Work.

Girders with riveted joints, with various forms of welded joints and welded girders built up from plates, are being made for tests. It is also hoped to test butt-welds in 3-inch plates, similar to those proposed for high-pressure boiler-drums.

Results of the Tests, and the Factor of Safety.

From the results of these tests, together with many others carried out by one of the Authors on welded plates, it is possible to give a simple diagram from which, assuming certain dead-load stresses in a girder and certain repeated-stress loadings, due to travelling loads and vibrations (these are particularly important in railway structures over which steam locomotives pass [3]), the safe repeated load that can be impressed upon a dead load can approximately be determined. The minimum stress to which the girder is to be subjected should be plotted as abcissæ, *Fig. 31* (p. 144), and the range of stress as ordinates. In the tests described the minimum stress was zero. Then OA, OB, and OC are the safe ranges of stress for zero minimum stress for the welded mild-steel girder, the drilled girder and the undrilled girder respectively. From these points straight lines should be drawn to a point of minimum stress equal to the breaking strength of the plates. This was about 27 tons per square inch. The strength of the weld-metal varied from 20 to 29·5 tons per square inch, but the former

[1] Footnote ([3]) p. 119.

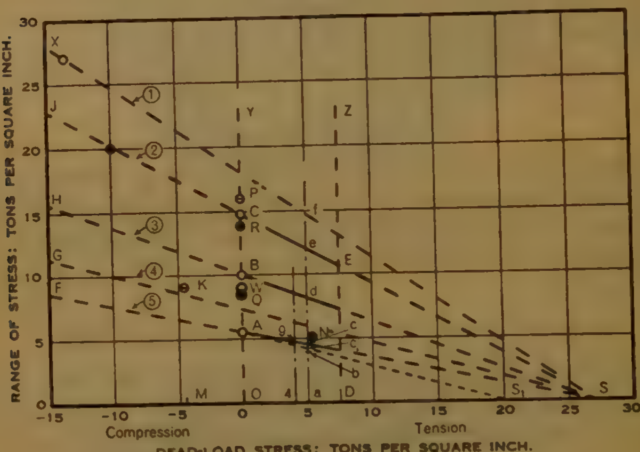
[2] Second Report of Welding Research Committee. Proc. Inst. Mech. E., vol. 133 (1936), p. 5.

[3] C. E. Inglis, "A Mathematical Treatise on Vibrations in Railway Bridges." Cambridge, 1934.

—— "Impact in Railway Bridges." Minutes of Proceedings Inst. C.E., vol. 234 (1931–32, Part 2), p. 358.

value was from a bad specimen. Through the points A, B, and C, straight lines are drawn to the point S and produced to F, H, and J. Then the safe range of stress for a very large number of repetitions of stress is given approximately by JS for the undrilled girder, HS for the girder with holes in the flanges, and FS for the rather bad batch of welded joists. For actual cases only a short length of these lines applies, say, between OY and DZ, and only a small difference is made by changing the forms of the curves CS, BS, and AS. If the

Fig. 31.



SAFE RANGES OF STRESS FOR VARIABLE MINIMUM STRESS.

①···Turned mild-steel specimens
②··Rolled-steel joists and some welds
③···Rolled-steel joists with holes in the tension-flange
④···Rather indifferent welds
⑤· Butt-welded joists examination of which after fracture showed obvious defects

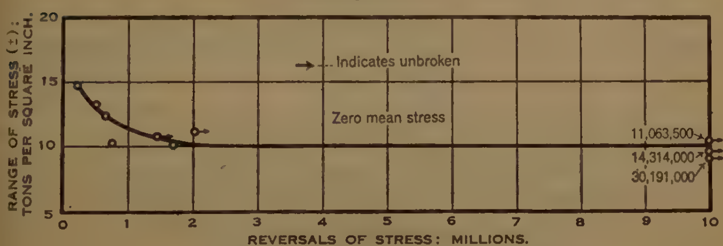
curve is assumed to be AS_1 for the welded joint, the difference of AS and AS_1 between YO and ZD is not very marked. Let now Oa be any dead-load stress and ab the range of stress produced by travelling loads. Then the factor of safety for the girder with rivet-holes may be said to be approximately $\frac{ad}{ab}$ and for the badly-welded girder $\frac{ac}{ab}$ or $\frac{ac_1}{ab}$. The maximum stress in the girder is oa + ab = 9 tons per square inch. If the dead-load stress is 4 tons per square inch and the maximum stress is 9 tons per square inch, the factor of safety of this particular welded girder for 4×10^6 repetitions of stress is less than unity. Point K shows an experimental point from butt-welds, and SG shows a probable line for butt-welds of certain plates

tested by one of the Authors. He has also obtained [1] values approximately corresponding to the line SJ, and in some cases points lying well above SJ. *Fig. 32* shows results at zero mean stress from turned specimens of weld-metal having the composition :—

Carbon	0·10	per cent.
Manganese	0·53	,,
Sulphur	0·025	,,
Phosphorus	0·11	,,
Silicon	Trace.	

The weld-metal was laid down from electrodes on to a flat steel plate, into ingots approximately 12 inches by 1½ inch by 1¼ inch. From these ingots test-specimens 0·25 inch in diameter were prepared and tested in a " constant-bending-moment " machine.[2] The results of

Fig. 32.



the tests are shown in *Fig. 32*, and SJ (*Fig. 31*) shows a probable safe-range line for various minimum stresses for machined specimens of this weld-metal. SX shows approximately the safe-range line for mild-steel turned specimens having a tensile strength of 27 tons per square inch. Point N corresponds to results of tests on welded joints carried out by Dr.-Ing. H. E. Neese.[3] Points P and K are two examples of points obtained by one of the Authors from welded joints tested under bending cycles of stress.[4] Points Q and R were obtained from tests of riveted joints, stressed in tension only from zero minimum stress. In the experiments now in hand various types of riveted and welded joints will be compared, and it is hoped to be able to prepare welded joints that will be better able to resist repeated stresses than riveted joints.

[1] Second Report of Welding Research Committee. Proc. Inst. Mech. E., vol. 133 (1936), p. 5.

[2] Prof. F. C. Lea, " The Testing of Materials." Chairman's Address to Birmingham and District Association of Inst. C.E. *Engineering*, vol. cxvi (1923), p. 633.

[3] " Form of Welds and Fatigue Strength of Welded Joints." *The Welding Industry*, vol. iii (1935), p. 303.

[4] Footnote (3), p. 119.

Conclusions, and the Meaning of the Factor of Safety.

The results given in this preliminary Paper show that the static tests of materials and structural elements are by no means a safe guide to indicate the ranges of stress to which materials can be subjected, when the stresses are repeated. For example, a rolled-steel joist that is drilled and strengthened by the addition of plates to the tension-flange will be stiffened against deflexion, but may be weaker under repeated stresses. Welded joints in mild-steel plates can be made that will withstand repeated stresses as great as those that a black rolled mild-steel joist will withstand, but, as shown in the Paper, unless great care is taken with the welding, fatigue-ranges of the order of one-third of those resisted by the joist may be obtained from welded joints. Unless designers of welded joists that are to be subjected to repeated stresses carefully control both the design and the technique of making the joint, only low fatigue-ranges can be relied upon. If, however, the fatigue-range is kept below the line FS in *Fig. 31* (p. 144) for specified minimum stresses, many millions of repetitions can be relied upon without risk of fracture. From *Fig. 31* it is possible to give some specific meaning to the "factor of safety." If a mild steel is bright turned, then for a dead-load stress of 5 tons per square inch and a range of stress of 4 tons per square inch above the 5 tons per square inch, the factor of safety would be $\frac{af}{ab}$. A drilled flange of a girder may have a factor of safety $\frac{ad}{ab}$. For a welded joint, a good portion of the joint of which will have the same tensile strength as the mild steel, the factor of safety may be anything between $\frac{ae}{ab}$ and $\frac{ac}{ab}$ unless the technique and the design of the joint are carefully controlled.

The Paper is accompanied by twenty-one sheets of drawings and thirty-six photographs, from some of which the Figures in the text and the six pages of half-tones have been prepared.

Paper No. 5092.

"The Discharge of Small Submerged Sharp-Edged Orifices."

By RONALD JAMES CORNISH, M.Sc., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*) [1]

TABLE OF CONTENTS.

SYMBOLS AND DEFINITIONS OF TERMS.

Let h_1 denote the head above the centre of the orifice on the upstream side.
,, h_2 ,, ,, head above the centre of the orifice on the downstream side.
,, h ,, ,, effective head, $h_1 - h_2$.
,, Q ,, ,, volume discharged in unit time.
,, A ,, ,, area of the orifice.
,, d_m ,, ,, hydraulic mean depth of the orifice, which is (area)/(perimeter).
,, k ,, ,, coefficient of discharge, which is $Q/A\sqrt{2gh}$.
,, ν ,, ,, kinematic viscosity, which is (absolute viscosity)/(density).
,, R_e ,, ,, Reynolds number, which is $d_m\, Q/\nu A$, defined in terms of the hydraulic mean depth.

INTRODUCTION.

The work of previous investigators has shown that the discharge of a submerged (or "drowned") orifice is in general less than that of an orifice discharging freely under the same effective head. The object of the work described in the Paper was to discover whether the coefficient of discharge is affected by the value of the head on the downstream side. Some experiments on the effect of temperature-variation are also reported.

[1] Correspondence on this Paper can be accepted until the 15th February, 1938, and will be published in the Institution Journal for October, 1938.—ACTING SEC. INST. C.E.

Apparatus.

A tank 8 feet long by 2 feet 9 inches wide by 3 feet 9 inches deep was divided into two compartments by a vertical plate 3 feet from one end. The brass diaphragm containing the orifice under test was fixed in this dividing plate,[1] with its centre 6 inches above the bottom of the tank. Water, which was circulated by a centrifugal pump, was admitted at the end of the larger compartment, the level in which was controlled by a funnel overflow. After traversing a set of baffles and stilling plates of perforated zinc, the water was discharged through the orifice to the smaller compartment, the water-level in which was maintained by an adjustable weir-plate. In this way the downstream submergence, h_2, could be varied from about 2·3 inches to 27·3 inches. On the upstream side the head, h_1, was limited to about 37 inches, so that the maximum possible effective head diminished from about 35 inches to 8 inches as the downstream submergence increased from 2·3 inches to 27·3 inches. The scope of the experiments for the larger downstream submergences was therefore considerably reduced.

The heads h_1 and h_2 were measured by hook gauges. The gauge-rods were graduated in inches, and intermediate values were read by a micrometer working in conjunction with an adjustable collar. Effective heads of less than 1 inch could be read directly on the micrometers without reference to the graduations on the rods. The water-surfaces were very little disturbed by the flow, and it was only at high heads with the larger orifices that any difficulty was experienced in setting the gauges.

The orifice-plates were bevelled on the downstream side to give "sharp" edges; the dimensions of the orifices, as measured by a travelling microscope, are given in Table I.

Table I.

Nominal size of orifice.	Measured dimensions: centimetres.	Area A: square centimetres.	Hydraulic mean depth (d_m): centimetre.
¼ inch square.	0·637 by 0·636	0·405	0·159
½ inch square.	1·269 by 1·278	1·622	0·318
¾ inch square.	1·899 by 1·907	3·624	0·476
1 inch square.	2·555 by 2·544	6·500	0·637
1 inch by ½ inch rectangular.	2·517 by 1·263	3·179	0·420

[1] The method of fixing the orifice is described in a Paper by the Author, "The Influence of Capillarity on the Free Discharge of Sharp-edged Orifices." *Phil. Mag.*, vol. xxii (seventh series), 1936, p. 181.

The areas of the orifices were small compared with that of the cross-section of the tank, so that the correction for velocity-head was negligible. The discharge per second was obtained by weighing the quantity passed during a measured period. By means of immersion heaters, the temperature of the water could be raised to between 90° F. and 100° F. Assuming a coefficient of expansion for brass of 0·000031 per degree F., the corresponding change in the linear dimensions of the orifice was about 1 in 1,000. The increase of area was allowed for in calculating the coefficient of discharge and the Reynolds number.

RESULTS.

In Figs. 1 and 2, Plate 1, the coefficients of discharge k for the various orifices have been plotted against Reynolds numbers, R_e. Fig. 1 gives the results with water at room temperature, and Fig. 2 shows those with warm water. Different downstream submergences are indicated by different symbols. A large scale has been adopted for k, and it will be seen that reasonably concordant results were obtained.

Effect of Downstream Submergence.

From a consideration of the curves for any particular orifice in either Figs. 1 or 2, Plate 1, it is evident that, below a certain Reynolds number, the downstream submergence has a very marked effect on the coefficient of discharge. Although the variation of k with submergence for any given orifice is fairly systematic, the influence of small variations in other conditions, such as temperature, was so great that no definite relationship could be established between k and h_2. All that can be said is that, in the present state of knowledge, the submerged orifice is unreliable as a measuring device at Reynolds numbers less than about 3,500.

For higher Reynolds numbers the submergence has no measurable influence, and eventually k becomes sensibly constant. An average straight line parallel to the R_e axis has been drawn in this region, and these average values of k are summarized in Table II.

It may be pointed out here that the above value for the rectangular orifice was unaffected by rotating the orifice through 90 degrees. This agrees with the conclusions of previous investigators, who, however, were dealing with the case of free discharge.[1]

[1] E. S. Bellasis, "Hydraulics with Working Tables," p. 54. London, 1931.

TABLE II.

Nominal size of orifice.	Reynolds number.	Coefficient of discharge. Temperature of water. 44°F. to 57°F.	86°F. to 100°F.
¼ inch square.	Greater than 2,500	0·629	0·628
½ inch square.	,, ,, 4,000	0·629	0·626
¾ inch square.	,, ,, 7,000	0·625	0·619
1 inch square.	,, ,, 8,000	0·627	0·624
1 inch by ¼ inch rectangular.	,, ,, 5,000	0·627	—

Effect of Temperature.

There is a definite general tendency, especially marked at low Reynolds numbers, for the coefficient of discharge to be lower with warm water than with cold. As the results are plotted on a Reynolds-number basis, this difference should not exist, and it seems evident that some physical factor other than viscosity and density should be included. A similar discrepancy in the case of free discharge was satisfactorily explained [1] by taking surface-tension into account, but this cannot be introduced when the orifice is submerged. The experiments reported in this Paper are only sufficient to indicate the existence of a temperature-effect, and further work is needed before an attempt can be made to explain it.

Comparison with Results of Other Investigators.

Most published data on orifice-flow relate to free discharge, or to the loss of head at orifices in pipes; it is usually stated that the coefficient for submerged discharge is less than that for free discharge, and this is confirmed [2] by the present experiments. The difference between the two diminishes as the Reynolds number increases, and is probably negligible at high Reynolds numbers.

As far as the Author has been able to discover, the only previous Papers discussing the effect of downstream submergence are those by Professor A. H. Gibson, M. Inst. C.E.,[3] and Mr. L. R. Balch.[4] In

[1] R. J. Cornish, "The Influence of Capillarity on the Free Discharge of Sharp-edged Orifices." *Phil. Mag.*, vol. xxii (seventh series), 1936, p. 181.

[2] "Free discharge" curves for the orifices under consideration are given on pp. 184 and 185 of the Paper referred to in footnote (1) (above).

[3] "Experiments on the Coefficients of Discharge under Rectangular Sluice-Gates." Minutes of Proceedings Inst. C.E., vol. ccvii (1918–19, Part I), p. 427.

[4] "Investigation of Flow through Four-Inch Submerged Orifices and Tubes." Bulletin of the University of Wisconsin No. 700, p. 21 (1914).

neither Paper were temperatures given, but if normal room temperatures are assumed it appears that in Professor Gibson's experiments the Reynolds number ranged from about 7,000 to 60,000, and in Mr. Balch's from about 20,000 to 80,000. Professor Gibson experimented with an apparatus representing a sluice-gate in a flume 3 feet wide; the gate occupied the whole width of the flume, and the bottom of the orifice was at the same level as the bottom of the flume. He found that as the depth of downstream submergence increased the coefficient of discharge decreased, but th rate of decrease fell off rapidly with increased submergence, and the coefficient appeared to tend to a constant value.

Mr. Balch's experiments were carried out with a tube of length 1 inch, the cross-section being a square of side 4 inches. The tube discharged into a flume 4 feet wide, and the centre of the tube was about 18 inches above the bottom of the flume. With effective heads greater than 2 feet, k increased by 7 per cent. as the depth of downstream submergence increased from 1·27 foot to 1·5 foot, but there was only a slight further increase in k at a submergence of 1·78 foot.

The evidence of the two Papers cited is conflicting, but it must be remembered that the downstream conditions in the two cases were very different. In the Author's experiments, the unimportance of the depth of downstream submergence at high Reynolds numbers is probably due to the fact that the orifices were small compared with the cross-section of the body of water into which they discharged.

In considering the effect at low Reynolds numbers, mentioned on p. 149, reference might be made to the discharge of an orifice contained in a diaphragm in a pipe-line. For example, the late Mr. J. L. Hodgson, Assoc. M. Inst. C.E., reported[1] some experiments with circular orifices which show that, at Reynolds numbers below about 3,500, the curves for coefficient of discharge for different values of the ratio (diameter of orifice)/(diameter of pipe) diverge widely, although they coincide again at extremely low Reynolds numbers. This indicates that over a certain range the flow-conditions on the downstream side have an important influence on the discharge. A useful summary of the present state of knowledge is given by Dr. F.V.A.E. Engel and Mr. J.W.E. French.[2]

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[1] "The Laws of Similarity for Orifice and Nozzle Flows." Proc. Am. Soc. Mech. E., vol. 51 (Part I, 1929), p. 303.

[2] "Orifices for Flow Measurements." *Engineering*, vol. cxlii (1936) pp. 410 and 496.

Conclusions.

(1) At Reynolds numbers less than 3,500 the value of the downstream head has a considerable influence on the coefficient of discharge of a square submerged orifice, which is therefore unreliable as a measuring device within this range.

(2) At Reynolds numbers higher than 3,500 the value of the downstream head has a negligible effect, provided that the dimensions of the orifice are small compared with those of the cross-section of the channel into which it discharges.

(3) A rise in temperature from about 50°F. to about 95°F. results in a decrease in the coefficient of discharge corresponding to the same Reynolds number.

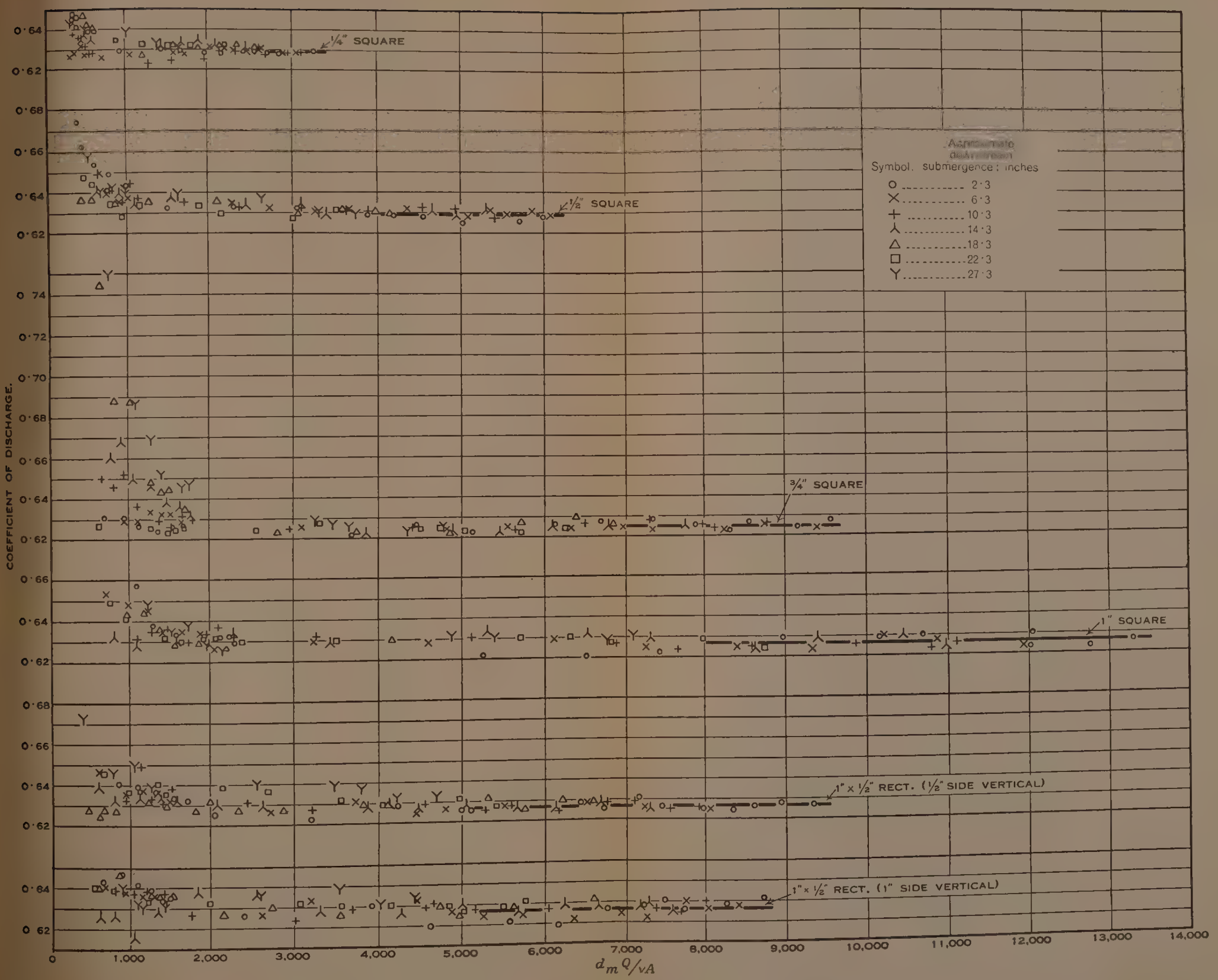
Acknowledgement.

The Author has pleasure in recording his thanks to his research assistants, Messrs. F. H. Chandler, M.Sc. Tech., and T. Coates, M.Sc. Tech., who carried out the experimental portion of the work.

The Paper is accompanied by two sheets of drawings, from which Plate 1 has been prepared.

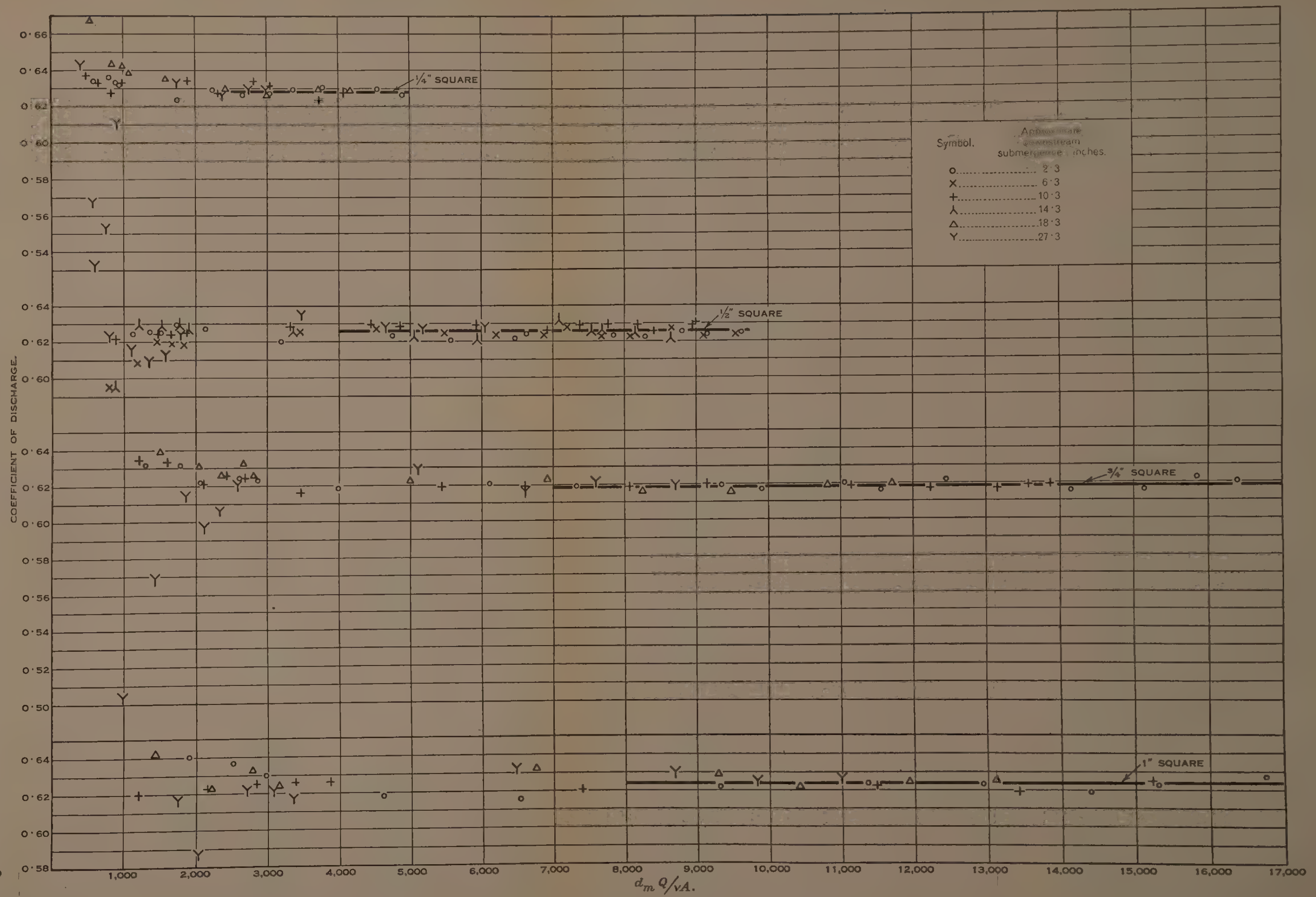
THE DISCHARGE OF SMALL SUBMERGED SHARP-EDGED ORIFICES.

FIG: 1.



RESULTS WITH WATER AT ROOM TEMPERATURE (44 °F. TO 57 °F.).

FIG: 2.



RESULTS WITH WARM WATER (86 °F. TO 100 °F.).

R. J. CORNISH.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Sub-Committee on Soil Corrosion of Cement Products.

THE formation of this Sub-Committee, under the Chairmanship of Mr. R. G. Hetherington, in place of the former Sub-Committee on the Effect of Soils containing Sulphate Salts on Concrete and Metal Pipes, was announced in the January, 1937, Journal.[1] It has now drawn up a comprehensive programme of research involving extensive field tests and large- and small-scale laboratory tests. This has been approved in principle by the main Committee on Soil Corrosion of Metals and Cement Products and by the Research Committee.

In view of the magnitude of the research, however, the estimated cost being of the order of £10,000 and the duration 10 years, it was decided that a preliminary investigation into procedure and method of execution should be carried out and a sum of £100 has been voted for this purpose. The Building Research Station is collaborating with The Institution in carrying this out. Visits are being paid to likely sites for field tests, the co-operation of the local authorities concerned sought, soil-samples taken and analysed, and estimates of cost drawn up.

RESEARCH WORK IN ENGINEERING AT UNIVERSITY COLLEGE, UNIVERSITY OF LONDON, OCTOBER, 1937.

The Faculty of Engineering at University College comprises departments of Civil, Mechanical, Electrical, Municipal and Chemical Engineering.

Civil and Municipal Engineering.

The study of the distribution of earth-pressures is rendered difficult by the relief of pressure afforded by a yielding surface. An attempt is being made to develop a cell suitable for measuring earth-pressures. The effect of temperature-variations on certain of the classification tests for soils is being studied with special reference to the London clays.

Experimental work with cements includes the determination of the amount hydrated in a mix with a view to investigating the effect

[1] Journal Inst. C.E., vol. 4 (1936–37), p. 488 (January, 1937)

of vibration on the proportions hydrated with Portland and alumina mixes; the effect of salinity and temperature of mixing water on the temperature-rises in alumina mixes; and the displacement of the specimen in permeability tests at pressures up to 300 lb. per square inch.

Researches on the treatment and purification of water cover the development of an apparatus for the measurements of turbidities and rates of coagulation, employing photometric cells; the effect of temperature and size of grain on rates of filtration; and the comparison of grain sizes found mechanically and by elutriation.

Mechanical Engineering.

The recent researches include an investigation into the accuracy of the spectrum-reversal method as applied to the measurement of the temperature of a flame, and the measurement of the temperature of the charge in an internal-combustion engine by the spectrum-reversal method. So far this test has been made only in the head of the cylinder, but tests are now proceeding to give the temperatures at different points in the stroke. The effect of detonation on flame-temperature has also been studied.

Tests have been completed recently on a new method of determining the rate of flame-travel, either across the head or down to the piston of a gas engine; this method will next be applied to petrol and oil engines.

A new form of grip has been devised for producing axial loading in tensile test specimens. This allows the yield point of a mild steel to be raised to about 90 per cent. of the maximum strength, an effect previously demonstrated at Professor Haigh's laboratory.

Electrical Engineering.

An investigation is being made into the measurement of the magneto-strictive effect in iron alloys. This is the small change in dimensions due to magnetization, the maximum value of which is about 30×10^{-7} of the length of the sample. A Lamb extensometer is used and an accurate curve is obtained showing the relation between change in dimensions and magnetizing force. The results obtained are compared with other methods from the points of view of accuracy and ease of working.

The noise emitted by transformers is being studied with a view to finding means of eliminating or reducing it. Model cores are energized in a sound-proof chamber and the sound analysed to give its sound-spectrum with a number of controlled variables.

Various researches are being conducted in electro-communication

work. Among these is a study of the secondary emission of electrons, a phenomenon made use of in the electron multiplier, where the effect can be multiplied a billion times. The magnetic properties of nickel-iron alloys at the ultra-high frequencies used in television are under investigation and research is being conducted in the design and performance of choke coils carrying an alternating current superimposed on a direct current. Another piece of research work concerns the reception of micro-waves having a wave-length of less than 60 cm. Recent research has been conducted on the validity of Ohm's law at very great current-densities.

Chemical Engineering.

Research work is being conducted on the processes of distillation and absorption; these include the obtaining of data for absorption in a grid-packed tower, the distribution of liquid in coke and ring packings, the rate of absorption of carbon monoxide by cuprous chloride solutions, the fundamental relationships between the physical properties of the liquid and vapour and the loading velocity in a column, and the rate of discharge of liquid over a sharp-edged circular weir in connexion with downtake-design in stills. A still has been constructed for separating the constituents of water, and considerable quantities of water rich in H_2O^{18} have been produced. Researches recently completed include measurements of the power required for emulsification and experiments with a new type of venturi tube.

The London Shellac Research Bureau is conducting work in the Laboratory dealing with industrial applications of shellac. Among other problems, the production of hard lac resins, the waterproofing of jute with shellac compositions and the spraying of shellac are being investigated.

The above researches are being carried out under the direction of Professor G. T. R. Hill, M.Sc., Professor of Engineering and Dean of the Faculty; Assistant Professor B. J. Lloyd-Evans, M.Sc.; Professor H. John Collins, M.Sc., Professor of Civil and Municipal Engineering; Professor R. O. Kapp, B.Sc., Professor of Electrical Engineering; and Professor H. E. Watson, D.Sc., Professor of Chemical Engineering.

NOTES ON RESEARCH PUBLICATIONS.

Measurement and Measuring and Recording Instruments.

The principles underlying the use of current-meters for discharge measurement are discussed in *Forschungsheft No. 385* (*Eng. Abs.* **76,** 11). The flow over a weir making an oblique angle with the axis of the channel is dealt with in *Comptes Rendus*, **204,** *1547* (*Eng. Abs.* **76,** 208). Constant temperature: a study of principles in electric thermostat design and a description of a mains-operated isothermal chamber constant to one-thousandth of a degree centigrade, is given in *J. Inst. Elec. Engineers*, **81,** *399*.

Engineering Materials: Properties and Testing.

The corrections to be applied to the general theorems relative to the resistance of materials when the displacement is not negligible are discussed in *Le Génie Civil*, **110,** *457* (*Eng. Abs.* **76,** 70).

Bricks, Cement, and Concrete.

A discussion on the testing of the weathering of building materials by the so-called crystallization test is given in *Mitteilungen des Technischen Versuchsamtes*, *1936*, **25,** *14*. An article on cement-content as the decisive factor for concrete quality, in which modern methods in the proportioning and preparation of mass concrete are reviewed with particular reference to dam construction, is contained in *Betong*, *1937* (*2*), *152*. The influence of the method of deposition of concrete upon its strength is discussed in *Bulletin Technique de la Suisse Romande*, **63,** *154* (*Eng. Abs.* **76,** 54).

Metals.

Papers on notched-bar impact testing were read at the meeting of the Joint Committee of Technical Institutions and Societies in Great Britain on Materials and their Testing, organized by the Manchester Association of Engineers, October, 1937. The influence of the mean stress of the cycle on the resistance of metals to corrosion fatigue is discussed in *J. Iron & Steel Inst.*, **135** (*1*) *293P*, and on *p. 315P* the effect of protective coatings on the corrosion-fatigue resistance of steel is considered.

The figure in heavy type is the number of the Volume; that in brackets the number of the Part; and that in italic type the number of the Page; in references to "Engineering Abstracts," the number of the Abstract is given.

Engineering Materials: Production, Manufacture, and Preservation.

An abstract of an article on the cleaning of natural and artificial stone is given in *Angewandte Chemie, 1937*, **50** (*31*), *634*. Problems of corrosion-prevention and corrosion-prevention technique are reviewed in *Tek. Tidskrift*, **67** (*17*) *Väg-och Vattenbyggnadskonst* (*4*), *37*.

Structures.

Framed Structures.

The elastic stability of a long and slightly-bent rectangular plate under uniform shear is considered in *Proc. Roy. Soc. Series A*, **162,** *62*. Notes on the stability of columns and beams are given in *The Structural Engineer*, **15,** *350*, and on *p. 403*, laterally-loaded struts are discussed, an alternative to Perry's approximation being given. Theoretical and experimental determinations of the stresses and deflexions in loaded rectangular plates on elastic foundations are given in *Iowa Engg. Expt. Stn. Bull. No. 135*. A method of solution of rigid frames of members of constant section by the theorem of joint translation is explained in *Univ. Washington Engg. Expt. Stn. Bull. No. 89*. A revision of the British Standards Specification based on the Code of Practice for the Use of Structural Steel in Building recommended by the Steel Structures Research Committee, has been issued by the British Standards Institution. Vessels under external pressure are dealt with in *Mechanical Engineering* (New York), **59,** *601* (*Eng. Abs.* **76,** 116). The results of tests on the holding strength of thick steel bars anchored in concrete blocks are given in *Bauing.*, **18,** *467* (*Eng. Abs.* **76,** 80). The calculation of stresses in reservoirs with plane walls is explained in *Annales des Ponts et Chaussées*, **107**-*i*, *538* (*Eng. Abs.* **76**, 76), and on *p. 618* (*Eng. Abs.* **76,** 77), calculations for a series of cylindrical reservoirs of similar proportions are considered.

Constructional Operations and Methods.

A description is given in *Eng. News-Record*, **119,** *220* (*Eng. Abs.* **76,** 102) of the by-passing of a fault in the construction of a tunnel.

Transformation, Transmission, and Distribution of Energy.

The work of the British Non-Ferrous Metals Research Association on condenser-tube corrosion and some trends of recent research are described in *Trans. Inst. Marine Engineers*, **49,** *171*. The silencing

of a four-stroke engine without power-loss is discussed in *Auto. Zeit.*, **40,** *383* (*Eng. Abs.* **76,** 133). A paper on automobile gears is contained in *J. Inst. Auto. Engineers*, **6** (*1*) *11*. A generator for a direct voltage of 8 million volts is described in *Zeit. Tech. Phys.*, **18,** *209* (*Eng. Abs.* **76,** 144). The question of high-voltage regulation is discussed in *J. Scientific Instruments*, **14,** *311*.

Mechanical Processes, Appliances, and Apparatus.

A paper on developments in resistance welding is given in *Welding Industry*, **5,** *297*.

A general discussion on Lubrication and Lubricants took place at the Institution of Mechanical Engineers in October, the papers read being grouped as follows: Group 1, Journal and thrust bearings; Group 2, Engine lubrication; Group 3, Industrial Applications; Group 4, Properties and testing.

Specialized Engineering Practice.

Transport.

The Department of Scientific and Industrial Research has issued *Road Research Technical Paper No. 2*, Studies in road friction II: an analysis of the factors affecting measurement. The papers presented at the 24th Annual Conference on Highway Engineering 1937, are contained in *Univ. Illinois Bull.*, **34,** *No. 76*. Laboratory, exposure, and simulated service tests of slow-curing asphalt are described in *Public Roads*, **18,** *85* (*Eng. Abs.* **76,** 21). The adsorption of bitumens by road aggregates is discussed in *J. Inst. Petroleum Technologists*, **23,** *491*. The results of tests on the effect of vibration upon the strength and uniformity of pavement concrete are given in *Public Roads*, **18,** *25*.

The Government of India Railway Department has published the 17th Report of the Bridge Standards Committee. The rail-joint problem on the Hälsingborg-Hässleholm railway is discussed in *Tek. Tidskrift* (*Mekanik*), **67,** *102* (*Eng. Abs.* **76,** 193). The third Progress Report of the joint investigation of fissures in railway rails has been published in *Univ. Illinois Bull.*, **34,** No. 88. *Indian Railway Board Technical Paper No. 299* (*Eng. Abs.* **76,** 192), deals with the determination of permissible speeds on curves.

A new device for the removal of sand from canals is described in *Le Génie Civil*, **111,** *39* (*Eng. Abs.* **76,** 210).

The following Aeronautical Research Committee Reports and Memoranda have been noted: *No. 1741*, Full-scale lift, drag, and

landing measurements of a monoplane fitted with a Zap flap; *No. 1765*, On Reynolds numbers of transition; *No. 1766*, Experiments on a sphere at critical Reynolds numbers; *No. 1767*, Abstract of a film illustrating the theory of flight; *No. 1769*, Induced drag due to washout; *No. 1774*, Aerodynamic characteristics of tapered wings with flaps and slots. In *J. Roy. Aero. Soc.*, **41,** *864*, is a paper on the art of dynamometry, with particular reference to the measurement of engine-power in flight; on *p. 921* the two-dimensional problem of wing vibration is discussed; and in this connexion on *p. 945* is given the consideration of internal damping in the two-dimensional problem of wing vibration. In *Engineering Journal (Canada) Aero. Section, Reprint No. 8, August*, 1937, the factors affecting the success of the North Atlantic air service London–Montreal are considered. The following National Advisory Committee for Aeronautics (U.S.) Reports have been noted: *No. 592*, Full-scale tests of N.A.C.A. cowlings; *No. 594*, Characteristics of six propellers, including the high-speed range; *No. 597*, Air propellers in yaw; *No. 603*, Wind-tunnel investigation of wings with ordinary ailerons and full-span external-airfoil flaps. The following articles are contained in *Luftfahrt.* **14**: *p. 325* (*Eng. Abs.* **76,** 233), Single-stage axial-flow fans; p. 347 (Eng. Abs. **76,** 205), The interaction between aeroplane wings and bodies; *p. 356* (*Eng. Abs.* **76,** 71), The crippling of curved tension-field girders; *p. 361* (*Eng. Abs.* **76,** 203), The lifting force acting on an oscillating control surface. In *Aéronautique*, **19** (*Aérotech.* **16**), *61* (*Eng. Abs.* **76,** 206) consideration is given to the hydrodynamic lift from immersed wings.

Water-Supply and Sewage-Disposal.

The Ministry of Health and the Scottish Office have issued the *Second Annual Report, 1936–37*, of the Inland Water Survey Committee. A method of calculation of the reinforcement at pipe junctions is given in *Rev. Gén. Hydraulique*, **3,** *70* (*Eng. Abs.* **76,** 212). A progress report of the American Research Committee on Grounding of Electric Current is contained in *J. Am. Waterworks Assoc.*, **29,** *1223*. The application of the theory of the triangle of velocities to the analysis of experimental results obtained from helical turbines and pumps is discussed in *Le Génie Civil*, **110,** *108* (*Eng. Abs.* **76,** 105), and an experimental study of the laws of similitude of hydraulic turbines in *Rev. Gén. Hydraulique*, **3,** *3* (*Eng. Abs.* **76,** 106). The Department of Scientific and Industrial Research has issued *Water Pollution Research Technical Paper No. 6. Survey of the River Tees, Part III, The Non-Tidal Reaches—Chemical and Biological.*

Mining.

The Lancashire and Cheshire Safety in Mines Research Committee has issued a Report on runaways on endless over-rope haulage, in *Trans. Inst. Mining Engineers,* **93,** *408.*

Ventilation.

Methods of dust-collection are described in *J. Inst. Heating and Ventilating Engineers,* **5,** *317.*

Telegraphy and Telephony.

In *Hochfrequenztechnik,* **49,** *199* (*Eng. Abs.* **76,** 235) there is an article on the high-frequency equivalent generator of a source of wireless interference; and in the same Journal, vol. **50,** the following articles have been noted: *p. 1* (*Eng. Abs.* **76,** 236), Frequency-control of transmitters by longitudinal oscillations of tourmaline crystals; *p. 18* (*Eng. Abs.* **76,** 234), An arc-generator of constant frequency; *p. 37* (*Eng. Abs.* **76,** 17), Impulse direction-finding.

Miscellaneous.

The report of a conference on the conduction of electricity in solids is given in *Proc. Phys. Soc.,* **49,** *Extra Part No. 274.* Research on gaseous dielectrics is described in *Archiv für Elektrotechnik,* **31,** *495* (*Eng. Abs.* **76,** 50).

ADDENDUM.

Journal Inst. C.E., vol. 5 (1936–37), p. 335 (March 1937).

The Author also wishes to acknowledge the facilities placed at his disposal by the Battersea Polytechnic in connexion with his research.

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